# CHAPTER 5
# DIAPHRAGMS

**5-1. Introduction**. This chapter prescribes the criteria for the design of horizontal diaphragms and horizontal bracing of buildings in seismic areas, indicates principles and factors governing the horizontal distribution of lateral forces and resistance to lateral forces, gives certain design data, and illustrates typical details of construction.

**5-2. General.**

*a. Function.* Floors and roofs, acting as diaphragms, are the horizontal resisting elements in a building. Diaphragms are subject to lateral forces due to their own weight plus the tributary weight of walls connected to them. The diaphragms distribute the lateral forces to the vertical elements, the shear walls or frames, which resist the lateral forces and transfer them to lower levels of the building and finally to the ground. If floors or roofs cannot be made strong enough, their diaphragm function can be accomplished by horizontal bracing. In an industrial building, horizontal bracing can be the only resisting element. Where there is a horizontal offset between resisting vertical elements above and below, the diaphragm transfers lateral forces between the. Diaphragms are treated in this chapter; the resisting vertical elements are treated in subsequent chapters.

*b. Horizontal elements.* There are two types of horizontal elements: diaphragms and horizontal bracing.

(1) *Diaphragms.* Usually the roof and the floors of the building perform the function of distributing lateral forces to the vertical resisting elements (such as walls and frames). These elements, called diaphragms, make use of their inherent strength and rigidity, supplemented, when needed, by chords and collectors. A diaphragm is analogous to a plate girder laid in a horizontal plane (or inclined plane, in the case of a roof). The floor or roof deck functions as the girder web, resisting shear; the joists or beams function as web stiffeners; and the chords (peripheral beams or integral reinforcement) function as flanges, resisting flexural stresses (fig 5-1). A diaphragm may be constructed of any material of which a structural floor or roof is made. Some materials, such as cast-in-place reinforced concrete and structural steel, have well-established properties and present no problems for diaphragm design once the loading and reaction system is known. Other materials, such as wood sheathing and metal deck, have properties that are well established for vertical loads but not so well established for lateral loads. For these materials, tests have been required to demonstrate their ability to resist lateral forces. Moreover, where a diaphragm is made up of units such as sheets of plywood or metal deck, or precast concrete units, the characteristics of the diaphragm are, to a large degree, dependent upon the connections that join one unit to another and to the supporting members.

(2) *Horizontal bracing.* A horizontal bracing system may also be used as a diaphragm to transfer the horizontal forces to the vertical resisting elements. A horizontal bracing system may be of any approved material. A common system that is not recommended is the rod or angle bracing used in industrial buildings. The general layout of a bracing system and the sizing of members must be determined for each individual case in order to meet the requirements for load resistance and deformation control. The bracing system will be fully developed in both directions so that the bracing diagonals and chord members form complete horizontal trusses between vertical resisting elements (fig 5-2). Horizontal bracing systems will be designed using diaphragm design principles.

*c. Seismic loadings.*

(1) *Principal load.* Floors and roofs used as diaphragms will be designed to resist the lateral forces specified in SEAOC 1H2j, acting in any horizontal direction. This load is for the diaphragm as a whole and its connections to the resisting shear walls or frames. The load represents the inertia forces originating from the weight of the diaphragm and the walls and other elements attached thereto.

(2) *Transfer forces.* The diaphragm design will also provide for transfer of forces from vertical resisting elements above to vertical resisting elements below where there is an offset or change of stiffness between the upper and lower walls (SEAOC 1H2j(1)(b)). (The designer is urged to be cautious in the use of computer analysis of structures with offsets in the vertical elements.)

(3) *Collectors.* The diaphragm will also be provided with collectors. These members collect diaphragm forces that are distributed along a portion of the depth of the diaphragm and transfer them as a concentrated load to a resisting wall or frame (SEAOC 1H2f).

(4) *Deformational compatibility.* When diaphragms move, they carry with them the tops of

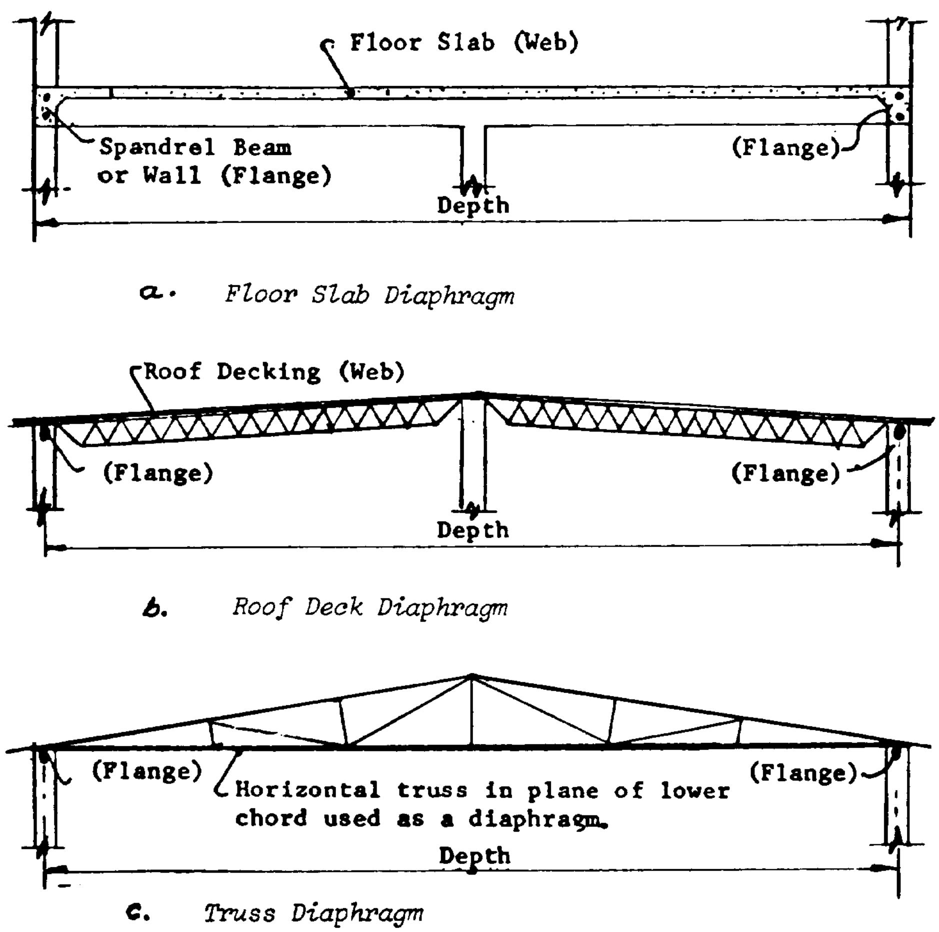



*Figure 5–1. Diaphragms.*

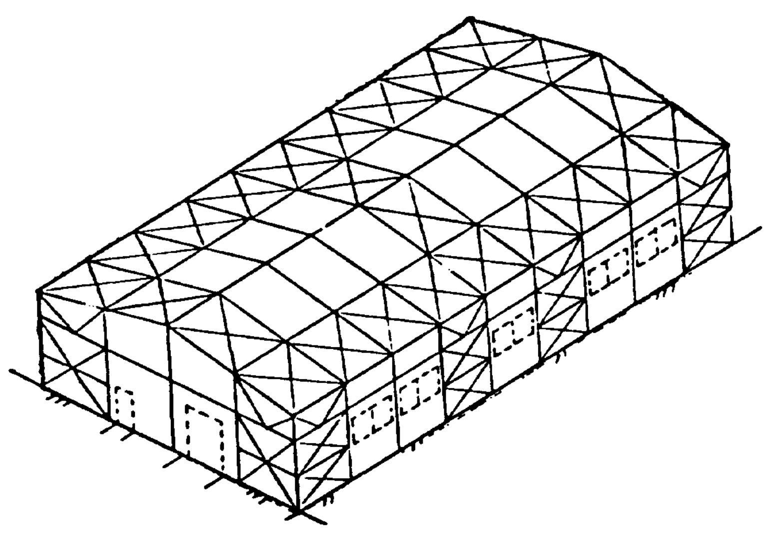


*Figure 5–2. Bracing an industrial building.*

other vertical elements, ones that are not part of the lateral force resisting system. Such elements are subject to the requirement for deformational compatibility. Columns, for example, are subject to side-sway moments when the diaphragms at the top and the bottom of the column have a relative displacement, and the columns have sufficient ultimate strength to sustain such moments when the diaphragm displacement is $3(R_w/8)$ times the displacements due to design forces for the diaphragm.

*d. Diaphragm selection.* Roofs and floors are usually utilized as diaphragms; therefore, diaphragm requirements must be considered when the overall structural system is selected. The diaphragm system must be compatible with the criteria governing the vertical load carrying capacities and the fire-resistant qualities. Relative costs of various types of suitable diaphragms should be investigated to achieve the greatest economy. Special considerations for diaphragm selection are summarized below.

(1) *Metal building systems.* For buildings with vertical moment resisting frames in the transverse direction, the systems connecting these frames are only nominal bracing with little or no computed stress, since each frame can be designed to carry its tributary lateral force. However, in the longitudinal direction, where only the exterior walls resist seismic forces, the diaphragm must span from side wall to side wall. Tension-only bracing may be used only if the structure is very light.

(2) *Multistory frame structures.* For multistory buildings with moment resisting frames, diaphragms will be rigid enough to distribute horizontal forces and torsion in proportion to the relative rigidities of the frames. A more flexible diaphragm on such structures is to be avoided because it would permit portions of the building to vibrate out of phase with the rest of the structure.

## 5-3. Diaphragm flexibility.

*a. Relative flexibility.* The diaphragm design forces at any level include the forces tributary to the diaphragm and forces brought down to the diaphragm by vertical resisting elements above the diaphragm. The forces will be distributed to the various vertical elements below the diaphragm according to the relative flexibility of the diaphragm, i.e., the flexibility of the diaphragm relative to the flexibility of the vertical elements that provide the lateral support below the diaphragm. Diaphragms are classified as rigid or flexible. The difference between flexible and rigid diaphragms is illustrated in figure 5-3. As shown in figure 5-3, part a, the example building has two bays with shear walls of various rigidities.

(1) *Flexible diaphragm.* In one extreme case (fig 5-3, part b), the resisting vertical elements are perfectly rigid and have no deformation. In this case, diaphragm deflections occur between supports that do not move. The diaphragm deflections are in the same direction as the design loads. The diaphragm acts as like a continuous beam: the diaphragm moments and shears are obtained by familiar procedures for continuous beams. For a given direction of design forces, the vertical elements that are perpendicular to this direction move with the diaphragm and so are subject to out-of-plane deformations, but these elements take no part in the resistance to lateral forces: the lateral resistance is provided only by the elements parallel to the lateral forces. Of course, no wall or frame is perfectly rigid; for design purposes however, a diaphragm is assumed to fit this case if it is only relatively flexible compared with the walls or frames. SEAOC 1E6a provides the deflection criterion for determining when the diaphragm is to be considered flexible. This is illustrated in figure 5-4. The wood diaphragm is an example of a relatively flexible diaphragm. It is customary to design wooden diaphragms as flexible diaphragms whether the vertical elements are concrete walls, steel moment or braced frames, or plywood shear walls. Unfilled metal-deck roof diaphragms are also considered relatively flexible when the vertical elements are concrete walls or steel frames. Flexible diaphragms are usually designed by a simple procedure that ignores continuity in the beam and treats each diaphragm as a simple beam between resisting walls or frames. This is the “tributary area” model (fig 5-3, part c). In this method it is customary to proportion the chords for the simple-beam moments, but then to detail them to be continuous over the length of the building so as to preclude damage in the walls where the beam-end rotations of the simple beams would make the chord ends separate or compress. This procedure is simple and cost-effective because the simple-beam chord forces are generally larger than those that would be developed from a continuous-beam analysis.

(2) *Rigid diaphragm.* In the other extreme case, the diaphragm is perfecrtly rigid and has no deformations (fig 5-3, part d). In this case the distribution of forces in the diaphragm depends on the stiffness of the resisting vertical elements. In the example shown in figure 5-3, each end wall has twice the rigidity of the center wall: this means that the reaction at each end wall is twice that of the center wall, and the diaphragm shears are determined from these reactions. Just as there are no perfectly rigid walls, there are no perfectly rigid diaphragms, but relatively rigid diaphragms fit this case. Concrete diaphragms and concrete-filled metal-deck diaphragms are usually considered to be relatively rigid.

(3) *Diaphragm of intermediate flexibility.* Between the two extremes discussed above, there are cases where the diaphragm is flexible enough to have significant deflection under lateral load but is stiff enough to distribute a portion of its load to vertical elements in proportion to the rigidities of the vertical resisting elements. The action is analogous to a continuous concrete beam system of appreciable stiffness of yielding supports. The support reactions are dependent on the relative stiffnesses of both the diaphragm and the vertical

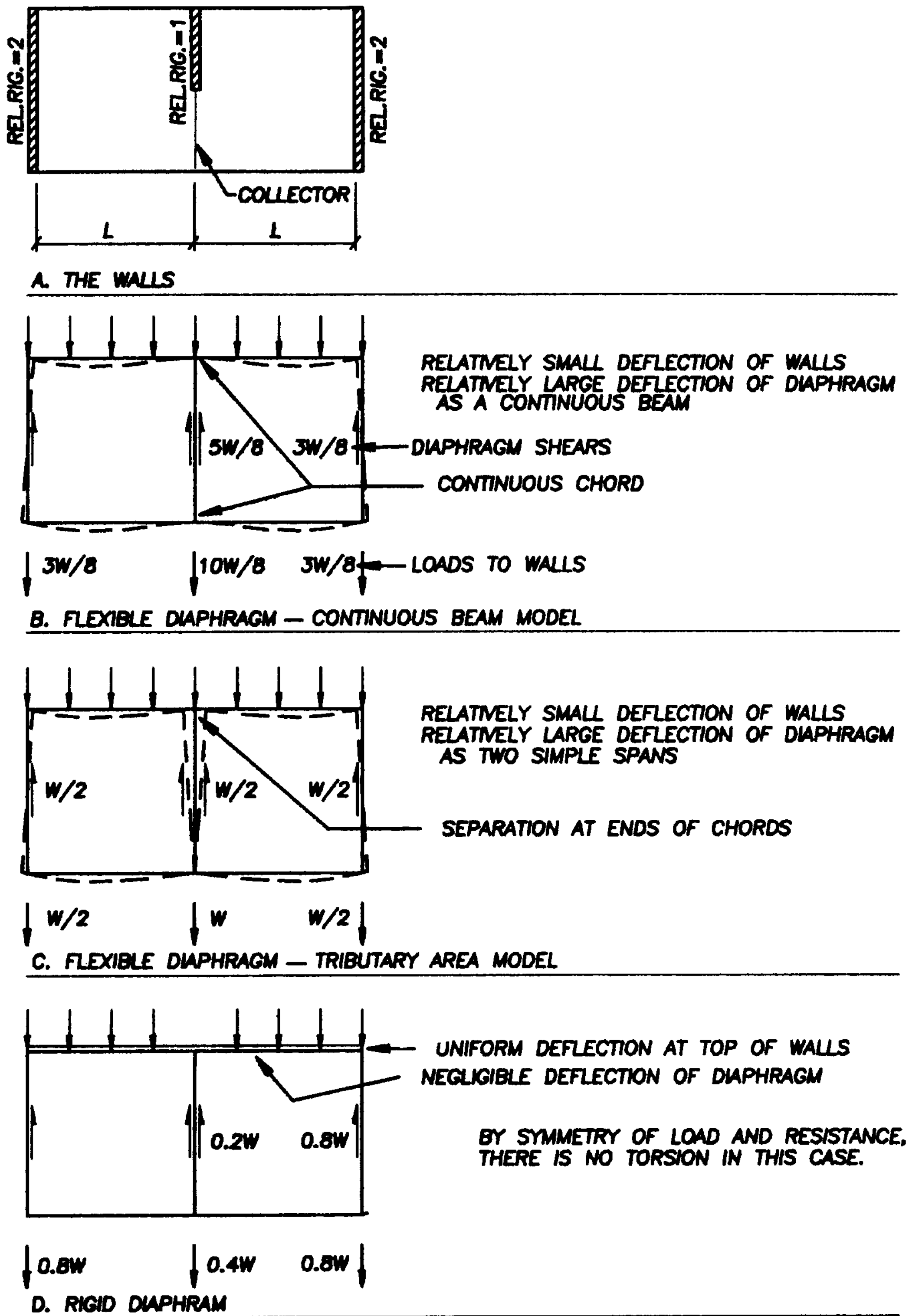



*Figure 5-3. Diaphragm flexibilities.*

elements. A rigorous analysis is usually very time-consuming and is seldom justifiable in terms of the doubtful accuracy of the results; at best, the results are no better than the assumptions (flexible and rigid) that must be made. In such cases the design can be based on two sets of assumptions that reasonably bracket the likely range of reactions and deflections.

*b. Rotation.* The example of figure 5-3 involves simple cases of diaphragms that have symmetry of load and reaction. In cases where there is a lack of symmetry, either in the load or the reaction, the

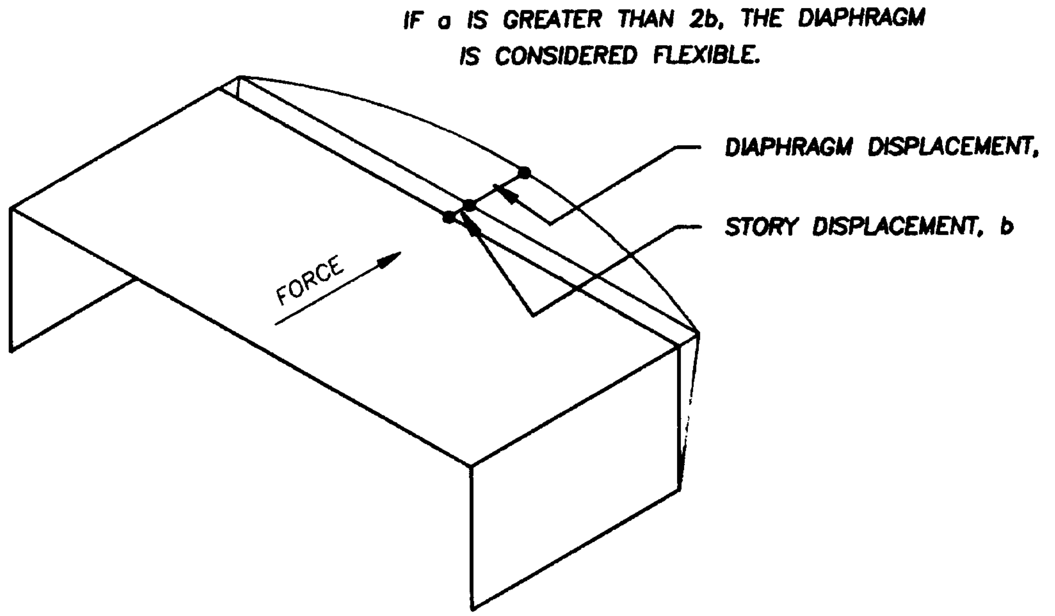



*Figure 5-4. Flexible diaphragm.*

diaphragm will experience a rotation. Rotation is of concern because it can lead to vertical instability. This is illustrated in the following cases: the cantilever diaphragm and the diaphragm supported on three sides.

(1) *Building with a cantilever diaphragm.* An example is shown in figure 5-5. The layout of the resisting walls is shown in figure 5-5, part a. If the backspan is flexible relative to the walls (fig 5-5, part b), the forces exerted on the backspan by the cantilever are resisted by walls B, C, and D, provided there are adequate collectors. If the backspan is relatively rigid (fig 5-5, part c), the load from the cantilever is resisted by all four walls (A, B, C, and D). A rigidity analysis is needed in order to determine the forces in the walls.

(2) *Building with walls on three sides.* An example is shown in figure 5-6. For transverse (north-south) forces (fig 5-6, part a), this is a simple case: because of symmetry of load and reactions, the end walls share the load equally. For longitudinal (east-west) forces (fig 5-6, part b), there is an eccentricity between the resultant of the load and the centerline of the one east-west resisting wall, wall C. The analysis is simplified by treating the load as a combination of the load, W, acting directly on the wall, and the couple M = WD/2 (fig 5-6, part c). The direct force induces a direct shear, W, on the diaphragm and a reaction, W, in wall C (fig 5-6, part d); the moment, M, is resisted by walls A and B (fig 5-6, part e), causing a counterclockwise rotation of the diaphragm. A particular concern with this type of building is the deflection at the corners at the open side. In figure 5-3, part a, there is an eastward deflection of the south edge of the diaphragm, and if this is excessive it can lead to vertical instability in the southwest and southeast corners.

*(a) Flexible diaphragm.* In an all-wood building, the concern about rotation is met by limitations on the size and proportions of the diaphragm. In buildings with walls of concrete or masonry, the greater weight causes greater concern for rotation, and there are special limitations on the diaphragms. The limitations are discussed in paragraph 5-10.

*(b) Rigid diaphragm.* If the diaphragm is rigid, the design of the building will consider the effects of torsion. The concept of orthogonality does not apply.

**5-4. Torsion.** Torsion, in a general sense, occurs in a building whenever the location of the resultant of the lateral forces, i.e., the center of mass, cm, at and above a given level does not coincide with the center of rigidity, cr, of the vertical resisting elements at that level. If the resisting elements have different deflections, the diaphragm will rotate. Torsion, in this general sense of rotation, occurs regardless of the stiffness properties of the diaphragms and the walls or frames. For purposes of design, however, the procedure for dealing with torsion does depend on these stiffness properties.

*a. Flexible diaphragms.* Flexible diaphragms such as wooden diaphragms can rotate but cannot develop torsional shears. For example, a single-span diaphragm with a relatively stiff shear wall at one end and a more flexible frame at the other end will rotate because the two resisting elements have

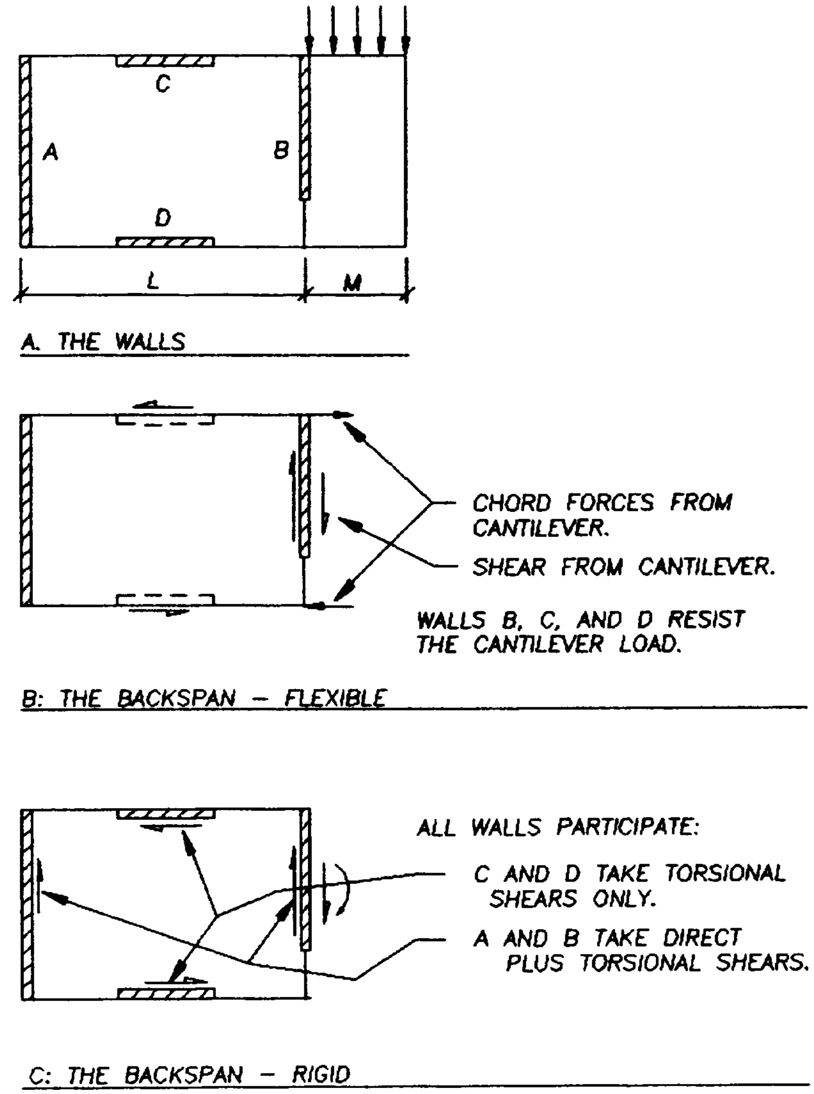



*Figure 5-5. Cantilever diaphragm.*

have different deflections. Flexible diaphragms, however, are considered incapable of inducing forces in the walls or frames that are perpendicular to the direction of the design forces; i.e., flexible diaphragms are said to be incapable of taking torsional moments. All of the lateral load is taken by the walls that are parallel to the lateral forces; none is taken by the other walls. (The building with walls on three sides is a special case and entails special limitations, as discussed above.) Lateral loads are usually distributed to the resisting walls by using the continuous beam analogy. There is no rigidity analysis, no calculation of the cm and the cr. If there are uncertainties about the locations of the loads and the rigidities of the structural elements, the design can be adjusted to bracket the range of possibilities.

*b. Rigid diaphragms.* When rigid diaphragms rotate, they develop shears in all of the vertical resisting elements. In the example of figure 5-7 there is an eccentricity in both directions, and all five walls develop resisting forces via the diaphragm.

*c. Deformational compatibility.* When a diaphragm rotates, whether it is rigid or flexible, it causes displacements in all elements attached to it. For example, the top of a column will be displaced with respect to the bottom. Such displacements must be recognized and addressed. The design condition is covered by SEAOC 1H2d.

*d. Flexibility criterion.* Provision for torsional moment is required only where diaphragms are not flexible. The criterion for flexibility (SEAOC 1E6a) is illustrated in figure 5-4.

*e. Analysis for torsion.* The method of determining torsional forces is indicated in figure 5-7. The diaphragm load, which acts through the cm, is replaced by an equivalent set of new forces. By adding equal and opposite forces at the cr, the diaphragm load can now be described as a combi-

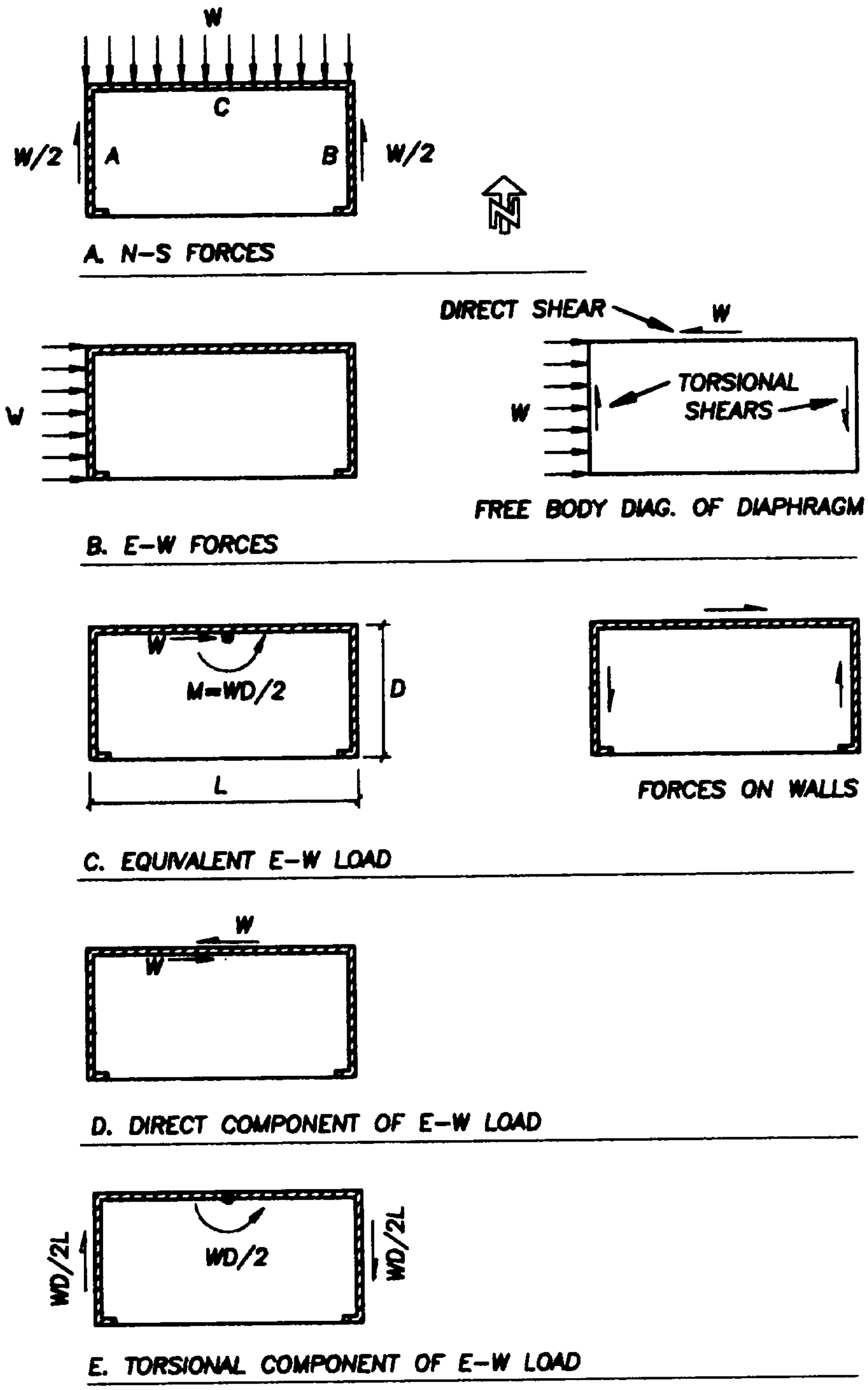



*Figure 5-6. Building with walls on three sides.*

nation of a force component, $F_{px}$, (which acts through the cr) and a moment component (which is formed by the couple of the two remaining forces $F_{px}$ separated by the eccentricity e). The moment, called the torsional moment, $M_T$, is equal to $F_T$ times e. The torsional moment is often called the "calculated" torsion because it is based on a calculated eccentricity; also this name distinguishes it from the "accidental" torsion which is described below. In the modified loading, the force $F_{px}$ acts through the cr instead of the cm; therefore, it causes no rotation and it is distributed to the walls which are parallel to $F_{px}$ in proportion to their relative rigidities. The torsional moment is resolved into a set of equivalent wall forces by a procedure which is similar to that used for finding forces on bolts in an eccentrically loaded group of bolts. The formula is analogous to the torsion formula $\tau = Tc/J$. Thus the torsional shear forces can be expressed by the formula $F_t = M_T kd/\Sigma kd^2$,

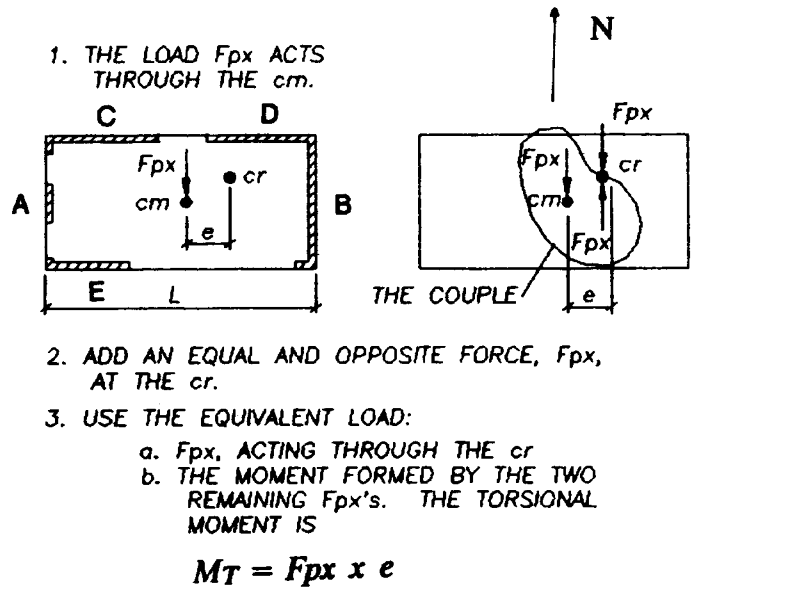



*Figure 5-7. Calculated torsion.*

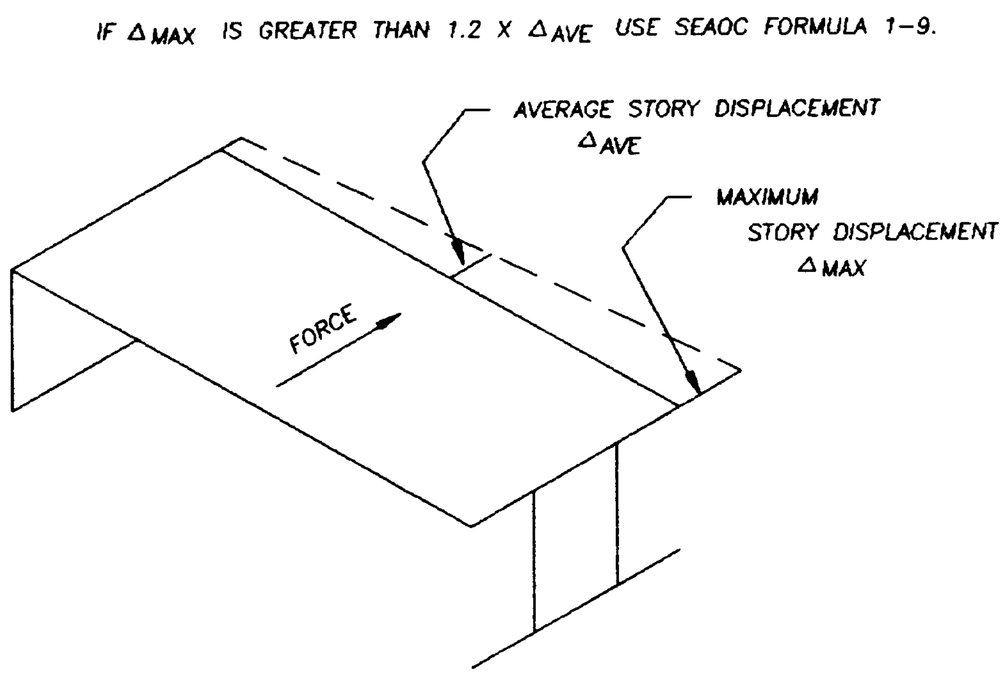



*Figure 5-8. Accidental torsion amplification factor.*

where k is the stiffness of a vertical resisting element, d is the distance of the element from the center of rigidity, and $\Sigma kd^2$ represents the polar moment of inertia. For the wall forces, the direct components due to $F_{px}$ at the cr are combined with the torsional components due to $M_T$. In the example of figure 5-7, the torsional moment is counter-clockwise and the diaphragm rotation will be counterclockwise around the cr. The direct component of the load is shared by walls A and B, while the torsional component of the load is resisted by walls A, B, D, C, and E. Where the direct and torsional components of wall force are in the same direction, as in wall A, the torsional component adds to the direct component; where the torsional component is opposite to the direct component, as in wall B, the torsional component subtracts from the direct. Walls C, D, and E carry only torsional components; in fact, their design will most likely be governed by direct forces in the east-west direction.

*f. Accidental torsion.* Accidental torsion is intended to account for uncertainties in the calculation of the locations of the cm and the cr. The accidental torsional moment, $M_A$, is obtained using an eccentricity, $e_{acc}$, equal to 5% of the building dimension perpendicular to the direction of the lateral forces (SEAOC 1E6); in other words, $M_A = F_{px} \times e_{acc}$ For the example of figure 5-7, the accidental torsion for forces in the north-south direction is $M_T = F_{px} \times 0.05L$. In hand calculations, $M_A$ is treated like $M_T$ except that absolute values of the resulting forces are used so that the accidental torsion increases the total design force for all walls. In computer calculations, the accidental torsion may be handled by running one analysis, using for eccentricity the calculated eccentricity plus the accidental eccentricity, then running a second analysis, using the calculated minus the accidental eccentricity, and finally, selecting the larger forces from the two cases.

*g. Amplification of accidental torsion.* When a torsional irregularity exists, the accidental torsion may be required to be increased. See SEAOC 1E6d and figure 5-8.

**5-5. Flexibility limitations.** The deflecting diaphragm imposes out-of-plane distortions on the walls that are perpendicular to the direction of lateral force. These distortions are controlled by proper attention to the flexibility of the diaphragm. A diaphragm will be designed to provide such stiffness that walls and other vertical elements laterally supported by the diaphragm can safely sustain the stresses induced by the response of the diaphragm to seismic motion.

*a. Empirical rules.* Direct design is not feasible because of the difficulty of making reliable calculations of the diaphragm deflections; instead, diaphragms are usually proportioned by empirical rules. The design requirement is considered to be met if the diaphragm conforms to the span and span/depth limitations of table 5-1. These limitations are intended as a guide for ordinary buildings. Buildings with unusual features should be treated with caution. The limits of table 5-1 may be exceeded, but only when justified by a reliable evaluation of the strength and stiffness characteristics of the diaphragm. For use of table 5-1, the flexibility category in the first column of the table can be determined with little or no calculation: concrete diaphragms are rigid; gypsum diaphragms are semirigid; metal deck diaphragms can be semirigid, semiflexible, or flexible; plywood diaphragms can be very flexible, flexible, or semiflexible; special diaphragms of diagonal wood sheathing are flexible; and conventional diaphragms of diagonal wood sheathing and diaphragms of straight wood sheathing are very flexible. (Very flexible diaphragms are seldom used in new construction because of their small capacities.) Each flexibility category of table 5-1 is associated with a range of values for the flexibility factor, F, and criteria for stiffness are specified in terms of the F-values given in the second column of table 5-1. When a value of F is needed in order to determine a flexibility category, it is obtained

| Flexibility Category | F | Allowable Span of Diaphragm (feet) | Diaphragm Span/Depth Limitations | |
|---|---|---|---|---|
| | | | Concrete or Masonry Walls[1] | Other Walls |
| Very Flexible[2] | Over 150 | 50 | Not to be used | 2:1 |
| Flexible | 70-150 | 100 | 2:1 | 3:1 |
| Semi-flexible | 10-70 | 200 | 2½:1 | 4:1 |
| Semi-rigid | 1-10 | 300 | 3:1 | 4:1 |
| Rigid | Less than 1 | 400 | 4:1 | 4:1 |

**Notes:**

[1]Walls in concrete and unit-masonry are classified as brittle; in all cases, check allowable drift before selecting type of diaphragm.
[2]For Zones 1 and 2, diagonally sheathed and plywood diaphragms in the "Very Flexible" category may be used for lateral support of masonry and concrete walls in one-story buildings where the diaphragm is not required to act in rotation.

*Table 5-1. Flexibility limitation on diaphragms.*

by procedures presented in the following section. Given either the flexibility category or the F-value, the maximum span is obtained from the third column of table 5-1, and span/depth limitations are given in the fourth column.

*b. F-factor.* When an F-factor is needed, it will be calculated by the following procedure. The flexibility factor, F, is equal to the average deflection in micro inches (millionths of an inch) of the diaphragm web per foot of span stressed with a shear of 1 pound per foot. Expressed as a formula, this becomes

$$F = \frac{\Delta_w \times 10^6}{q_{ave}\ L_1} \qquad \text{(eq 5-1)}$$

where

$L_1$ = distance in feet from the adjacent vertical resisting element (such as a shear wall) and the point to which the deflection is to be determined

$q_{ave}$= average shear in diaphragm in pounds per foot over length $L_1$

$\Delta_w$ = web component of diaphragm deflection

Note that for a diaphragm with a single span of length, L, and a uniformly distributed load, W, the average shear to be used in calculating $q_{AVE}$ is W/4, and $L_1$ = L/2. The procedures for calculating F, given in paragraphs 5-4 through 5-8, are summarized as follows

(1) *Concrete diaphragms.* The F-factor is obtained from conventional beam theory for shearing deflection. Using the procedure given in paragraph 5-7, one can calculate the shearing procedure given in paragraph 5-7, one can calculate the shearing deflection without having to know the beam theory. It should be noted that because concrete (and concrete-filled steel deck) diaphragms are generally rigid, deflections are seldom calculated.

(2) *Steel deck diaphragms.* The F-factor is obtained from formulas that were derived from tests. Values for F for common types of diaphragms are given in tables in this manual. Some manufacturers provide values in their literature. When the F-factor is not available, it can be calculated by using the procedures of paragraph 5-6. In most cases formulas for F have been published only in the literature of the companies supplying these materials. These formulas have usually been based on a limited number of tests and have been derived empirically to fit the test data applicable to them. As more and more tests were run, the formulas were altered to incorporate the new data. This has led to many somewhat similar formulas for identical diaphragm components supplied by different manufacturers. The formulas used in this manual have been developed by using as a basis all of the test data made available to the Tri-Service Seismic Design Committee at the time of the 1973 edition of this manual and may be subject to some revision in the future as new data are obtained.

(3) *Plywood diaphragms.* A formula for F is given in paragraph 5-10.

(4) *Wood-sheathed diaphragms.* Values for F are given in table 5-2.

| HORIZONTAL WOOD DIAPHRAGMS | F | ALLOWABLE SHEAR lbs./lin.ft.($q_D$) |
|---|---|---|
| 1" Straight Sheathing | 1,500 | 50 |
| 2" Straight Sheathing | 1,500 | 40 |
| Conventional 1" Diagonal Sheathing - 1x6 & 1x8 | 250 | 300 |
| Conventional 2" Diagonal Sheathing | 250 | 400 |
| Special Construction | 75 | 600 |
| Note: The allowable shears shown in Table are basic values to which the factors for species shown in Figure 6-19 will be applied. | | |

*Table 5-2. Flexibility and allowable shears.*

*c. Diaphragm deflections.* When a deflection calculation is needed, the following procedure will be used.

(1) *Deflection criterion.* The total deflection of the diaphragm under the prescribed static forces will be used as the criterion for the adequacy of the stiffness of a diaphragm. The limitation on deflection is the allowable amount prescribed for the relative deflection (drift) of the walls between the level of the diaphragm and the floor below. Refer to chapter 6 and figure 6-5.

(2) *Deflection calculations.* The total computed deflection of diaphragms ($\Delta_d$) under the prescribed static seismic forces consists of the sum of two components: the first component is the flexural deflection ($\Delta_f$); the second component is the shearing deflection ($\Delta_w$). When beams are designed, the flexural component is usually all that is calculated, but for diaphragms, which are like deep beams, the shearing component must be added to the flexural component.

*(a) Flexural component.* This is calculated in the same way as for any beam. For example, for a simple beam with uniform load, the flexural component is obtained from the familiar formula $\Delta_f = 5wL^4/384EI$. The only question is the value of the moment of inertia, I. For diaphragms whose webs have uniform properties in both directions (concrete or a flat steel plate) the moment of inertia is simply that of the diaphragm cross-section. For diaphragms of fluted steel deck, or diaphragms of wood, whose stiffness is influenced by nail slip and chord-joint slip, the flexural resistance of the diaphragm web is generally negligible and the moment of inertia is based on the properties of the diaphragm chords. For a diaphragm of depth D with chord members each having area A, the moment of inertia, I, equals $2A(D/2)^2$, or $AD^2/2$.

*(b) Shearing component.* If a reliable F-factor is known, the shearing component of deflection can be derived from equation 5-1 as follows:

$$\Delta_w = \frac{q_{ave} L_1 F}{10^6} \qquad \text{(eq 5-2)}$$

This equation is directly applicable to steel-deck diaphragms for which values of F are available and to concrete decks for which F is obtained by a simple calculation. If a reliable F-factor is not known, the calculation is based on conventional beam theory. For example, for a diaphragm with a single span of length, L, with a uniformly distributed load, W, the shearing deflection is $\Delta_w = \frac{\alpha WL}{8AG}$

where $\alpha$ is a form factor, A is the area of the web, and G is the shear modulus. Noting that $\frac{W}{A} = \frac{4q_{ave}}{t}$ where t is the thickness of the web, the formula for shearing deflection can be expressed as $\Delta_w = Q_{ave} L_1 \frac{(a)}{tG}$. As noted above, this is applicable only to webs of uniform properties. The procedure for concrete (given in para 5-7) is based on this equation, with $\alpha = 1.5$, $G = 0.4E$, $E = 33w^{1.5}\sqrt{f'_c}$, $\Delta_w = \frac{q_{ave} L_1}{8.8tw^{1.5}\sqrt{f'_c}}$ where t is the thickness of the slab in inches. This is equation 5-2 with F as given by equation 5-3.

**5-6. Design of diaphragms.** A deep-beam analogy is used in the design. Diaphragms are envisioned as deep beams with the web (decking or sheathing) resisting shear and the flanges (spandrel beams or other members) at the edges resisting the bending moment.

*a. Unit shears.* Diaphragm unit shears are obtained by dividing the diaphragm shear by the length or area of the web, and are expressed in pounds per foot (for wood or metal deck) or pounds per square inch (for concrete). These unit shears are checked against allowable values for the material. Webs of precast concrete units or metal-deck units will require details for joining the units to each other and to their supports so as to distribute shear forces.

*b. Flexure.* Diaphragm flexure is resisted by members called chords. The chords are often at the edges of the diaphragm but may be located elsewhere. The design force is obtained by dividing the diaphragm moment by the distance between the chords. The chords must be designed to resist direct tensile or compressive stresses, both in the members and in the splices at points of discontinuity. Usually chords are easily developed. In a concrete frame, continuous reinforcing in the edge beam can be used. In a steel frame building, the spandrel beams can be used as chords if they have adequate capacity and have adequate end connections where they would otherwise be interrupted by the columns; or special reinforcing can be placed in the slab. Chords need not actually be in the plane of the diaphragm as long as the chord forces can be developed between the diaphragm and the chord. For example, continuous chord reinforcing can be placed in walls or spandrels above or below the diaphragm. In masonry walls, the chord requirements tend to conflict with the control joint requirements. At bond beams, control joints will have to be dummy joints so that reinforcement can be continuous, and the marginal connections must be capable of resisting the flexural and shear stresses developed.

*c. Openings.* A diaphragm with openings such as cut-out areas for stairs or elevators will be treated as a plate girder with holes in the web. The diaphragm will be reinforced so that forces that develop on the sides of the opening can be developed back into the body of the diaphragm.

*d. L- and T-shaped buildings.* L- and T-shaped buildings will have the flange (chord) stresses developed through or into the heel of the L or T. This is analogous to a girder with a deep haunch.

## 5-7. Concrete diaphragms.

*a. General design criteria.* The criteria used to design concrete diaphragms will be ACI 318 as modified by SEAOC 3B. Concrete diaphragm webs will be designed as concrete slabs; the slab may be designed to support vertical loads between the framing members, or the slab itself may be supported by other vertical load carrying elements, such as precast concrete elements or steel decks. If shear is transferred from the diaphragm web to the framing members through steel deck fastenings, the design will conform to the requirements in paragraph 5-9.

*b. Span and anchorage requirements.* The following provisions are intended to prevent diaphragm buckling.

(1) *General.* Where reinforced concrete slabs are used as diaphragms to transfer lateral forces, the clear distance ($L_v$) between framing members or mechanical anchors shall not exceed 38 times the total thickness of the slab (t).

(2) *Cast-in-place concrete slabs not monolithic with supporting framing.* When concrete slabs are not monolithic with the supporting framing members (e.g., slabs on steel beams), the slab will be anchored by mechanical means at intervals not exceeding 4 feet on center along the length of the supporting member. This anchorage is not a computed item and should be similar to that shown in figure 5-9, detail A. For composite beams, anchorages provided in accordance with AISC provisions for composite construction will meet the requirements of this paragraph.

(3) Cast-in-place concrete diaphragms vertically supported by precast concrete slab units. If the slab is not supporting vertical loads but is supported by other vertical load carrying elements, mechanical anchorages will be provided at intervals not exceeding 38t. Thus, the provisions above will be satisfied by defining $L_v$ as the distance between the mechanical anchorages between the diaphragm slab and the vertical load carrying members. This mechanical anchorage can be provided by steel inserts or reinforcement, by bonded cast-in-place concrete lugs, or by bonded roughened surface, as shown in figure 5-10. Positive anchorage between cast-in-place concrete and the precast deck must be provided to transmit the lateral forces generated from the weights of the precast units to the cast-in-place concrete diaphragm and then to the main lateral force resisting system.

(4) *Precast concrete slab units.* If precast units are continuously bonded together as shown in figure 5-11, they may be considered concrete diaphragms and designed accordingly as described hereinbefore; see SEAOC 3E6 and 3E7. Intermittently bonded precast units or precast units with grouted shear keys will not be used as a diaphragm. In Seismic Zone 1 (fig 5-12), there is an exception permitting the use of hollow-core planks with grouted shear keys and the use of connectors, in lieu of continuous bonding, for precast concrete members. The exception is permitted if the following considerations and requirements are satisfied:

*(a)* Procedure conforms with PCI-MNL-120-seismic design requirements.

*(b)* Shear forces for diaphragm action can be effectively transmitted through the connectors. The shear is uniformly distributed throughout the depth or length of the diaphragm with reasonably spaced connectors rather than with a few which will have localized concentration of shear stresses.

*(c)* Connectors are designed for $3(R_w/8)$ times the prescribed shear force.

*(d)* Detailed structural calculations are made including the localized effects in concrete slabs attributed from these connectors.

*(e)* Sufficient details of connectors and embedded anchorage are provided to preclude construction deficiency.

(5) *Metal-formed deck.* Where metal deck is used as a form, the slab shall be governed by the requirements of paragraph (2) above. Refer to paragraph 5-9*d*, where the deck is used structurally.

*c. Special reinforcement.* Special diagonal reinforcement will be placed in corners of diaphragms, as indicated in figure 5-13. Typical chord reinforcement and connection details are shown in figure 5-14.

*d. Flexibility factor.* The web stiffness factor, F, will be determined by the following formula:

$$F = \frac{10^6}{8.5 \; t w^{1.5} \sqrt{f'_c}} \qquad \text{(eq 5-3)}$$

where

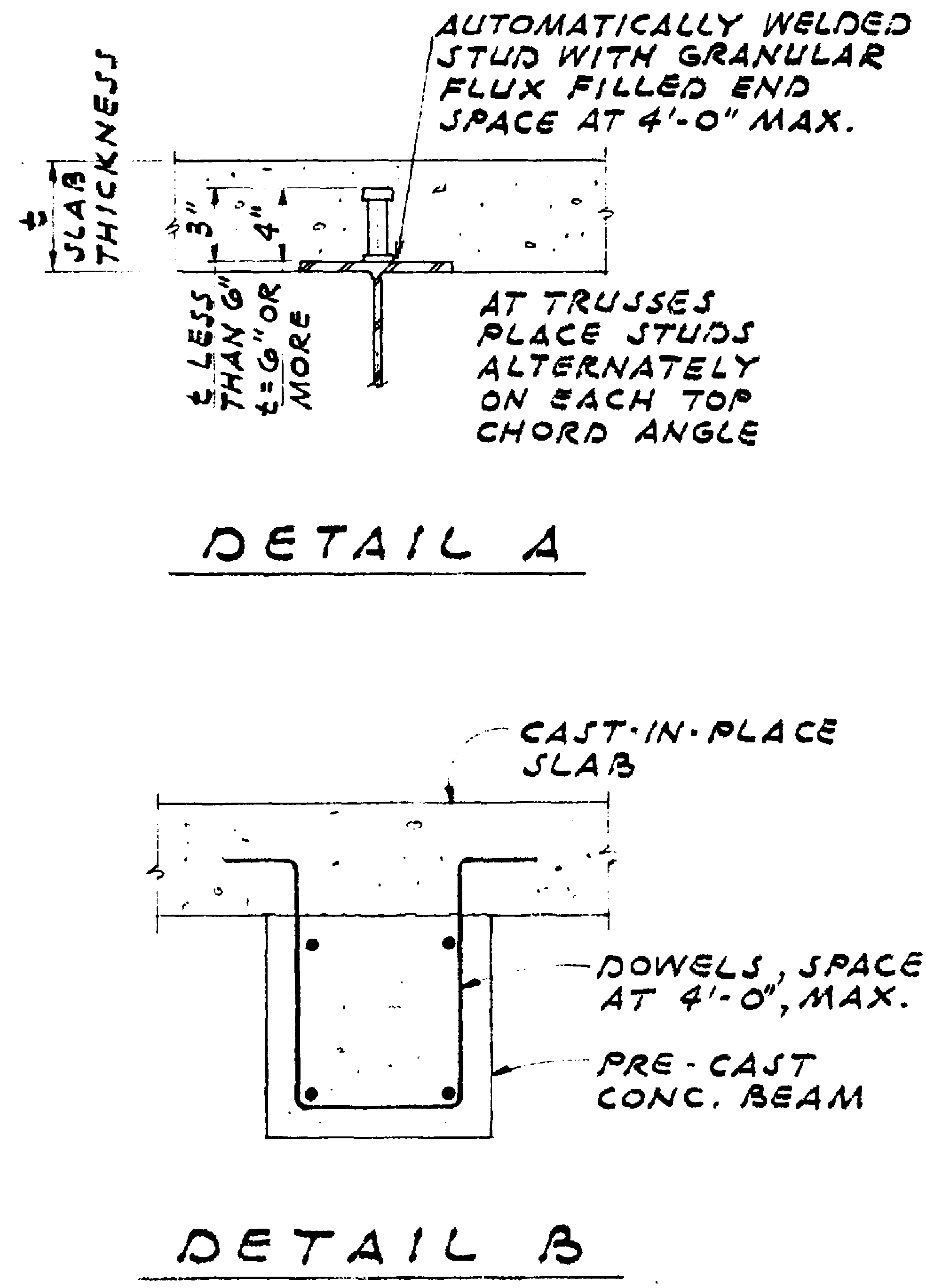
t = thickness of the slab in inches

w = weight of conrete in pounds per cubic foot, minimum value of w will be 90 pounds per cubic foot

$f'_c$ = compressive strength of concrete at 28 days in pounds per square inch

Diaphragms of this type are in the rigid category of stiffness and usually have a limitation only on deflection, as specified in SEAOC 1H2j.

*e. Electrical race ways.* The placement of electrical raceways in concrete topping slabs may make the slab ineffective as a diaphragm. The effect




Slabs Not Monolithic with Supporting Framing

*Figure 5-9. Anchorage of cast-in-place concrete slabs.*

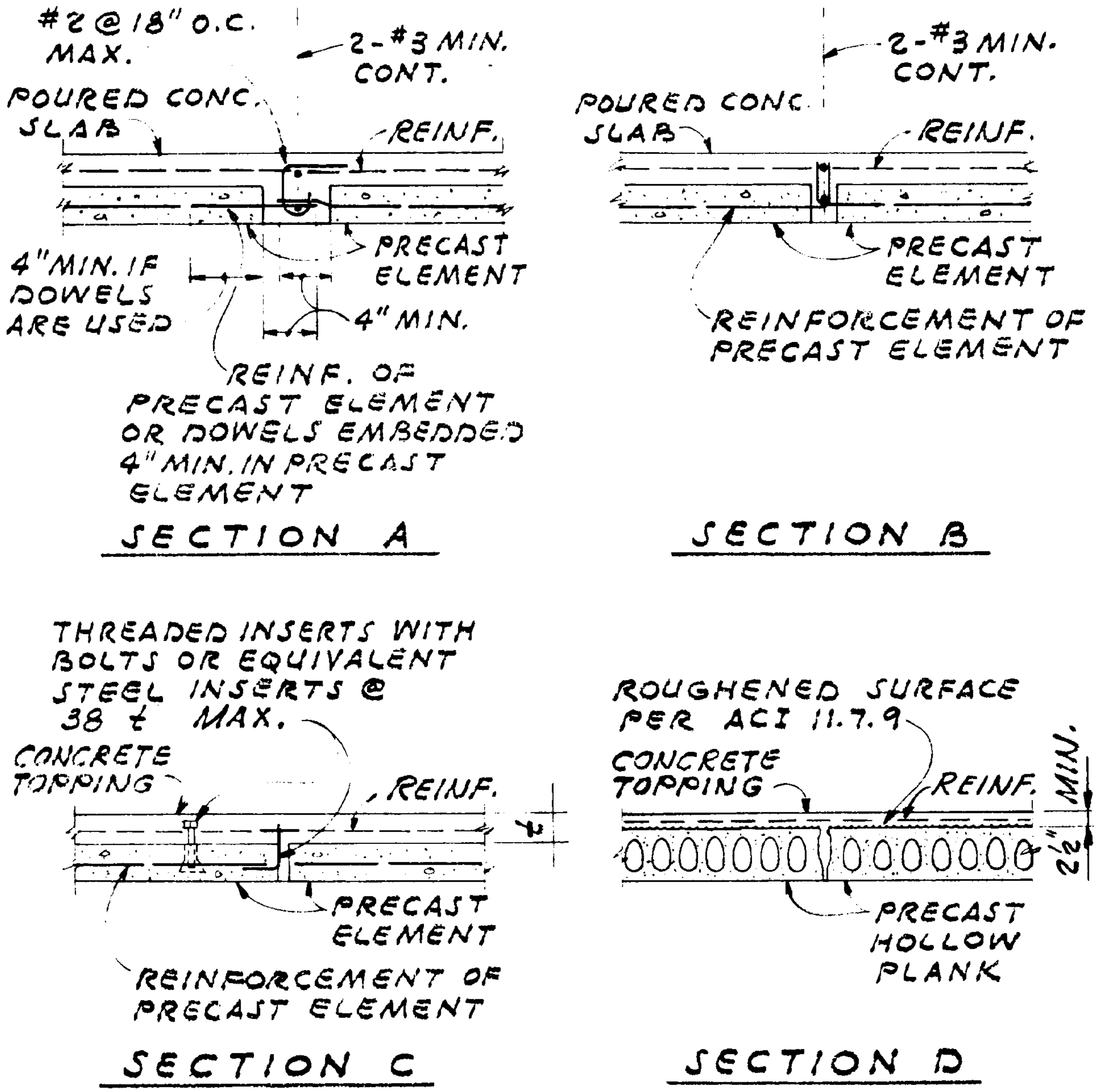



*Figure 5-10. Attachment of superimposed diaphragm slab* to *precast slab units.*

of the loss of concrete section will be considered. Coordination of structural diaphragm slab with electrical plans will be provided.

## 5-8. Gypsum diaphragms, cast-in-place.

*a.* General design criteria. The following criteria will be used to design cast-in-place gypsum diaphragms.

*b.* Shear capacity.

(1) The allowable diaphragm shear on poured gypsum concrete diaphragms will be as shown in tables 5-3, 5-4, and 5-5 for roof systems using subpurlins and welded wire fabric.

(2) In lieu of tables 5-3 and 5-4, the following formula will be used to determine the allowable shear of the diaphragm:

$$q_p = [.16 f_g t C_1 + 1{,}000(k_1 d_1 + k_2 d_2)] C_2 \qquad \text{(eq 5-4)}$$

where

- $q_D$ = allowable maximum shear per foot on diaphragm in pounds per linear foot, the one-third increase usually permitted to working stresses in seismic design is not applicable
- $f_g$ = oven-dry compressive strength of gypsum in pounds per square inch, as determined by tests conforming to ASTM C472-73
- $C_1$ = 1.0 for Class A gypsum concrete; 1.5 for Class B gypsum concrete
- $C_2$ = 1.4 for Class A gypsum concrete; 1.0 for Class B gypsum concrete
- t = thickness of gypsum between subpurlins, in inches

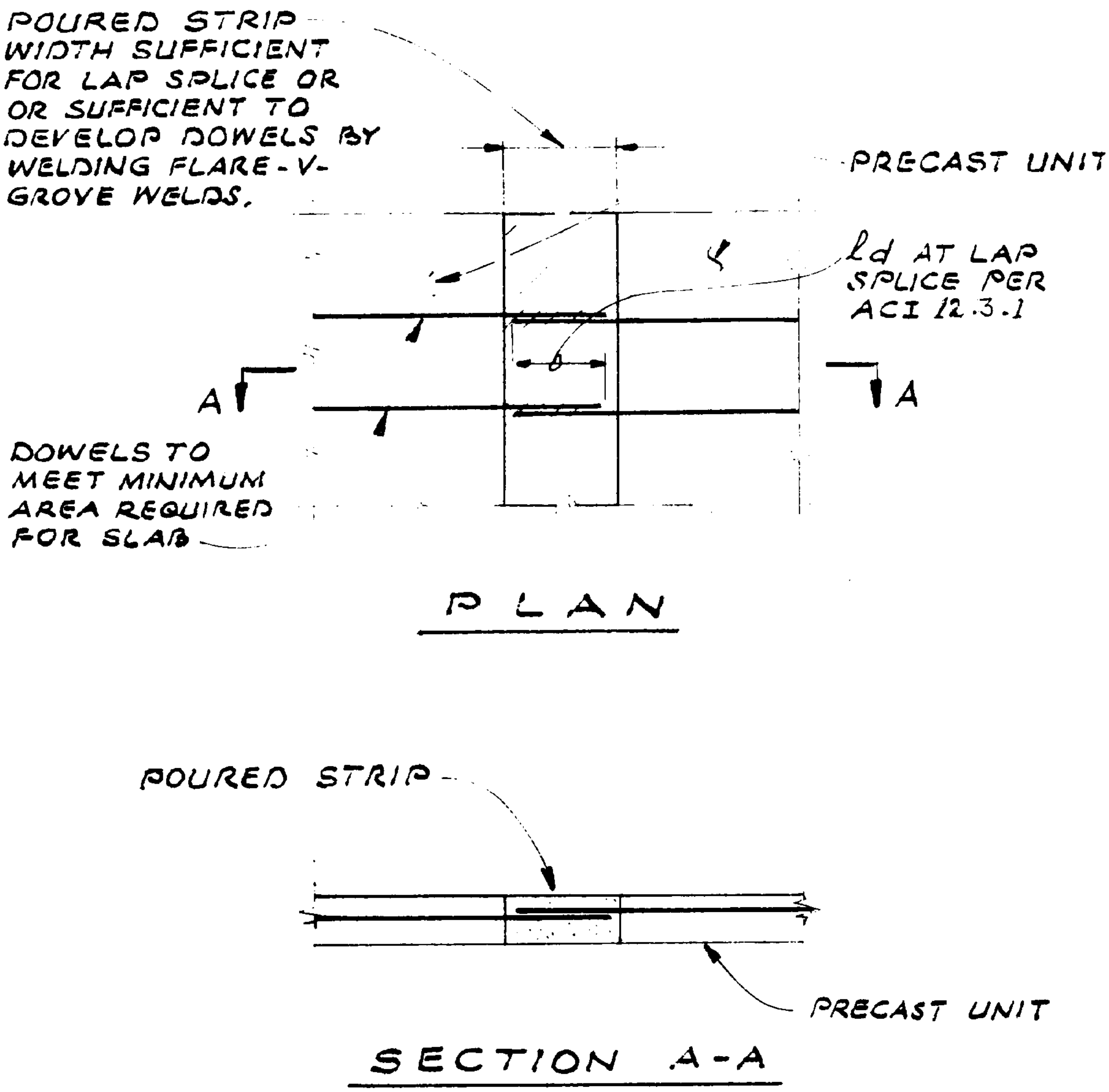



*Figure 5-11. Precast concrete diaphragms using precast units.*

$k_1$ = number of welded wire fabric wires per foot passing over subpurlins
$d_1$ = diameter of welded wire fabric wires passing over subpurlins. in inches
$k_2$ = number of welded wire fabric wires per foot parallel to subpurlins
$d_2$ = diameter in inches of welded wire fabric wires parallel to subpurlins

*c. Flexibility factor.* The factor F for determination of diaphragm stiffness and deflections will be determined by the formula

$$F = \frac{140}{\sqrt{q_D}} \quad \text{(eq 5-5)}$$

where

$q_D$ = the allowable shear specified in tables 5-3 and 5-4 or equation 5-4, in pounds per foot

This indicates that the diaphragm will be in the semirigid category; however, the span depth and span limitations of the semiflexible diaphragm should be used for this type of diaphragm.

*d. Typical details.* Refer to figure 5-15.

## 5-9. Steel deck diaphragms (single- and multiple-sheet decks).

*a. General design criteria.* The following criteria will be used to design steel deck diaphragms. The three general categories of steel deck diaphragms are Type A, Type B and decks with concrete fill. Design data from industry sources such as the Steel Deck Institute and the Research Reports of the International Conference of Building Officials may be used subject to the approval of the Agency Proponent.

(1) *Typical deck units and fastenings.* Deck units will be composed of a single fluted sheet or a combination of two or more sheets fastened together with welds. The special attachments used

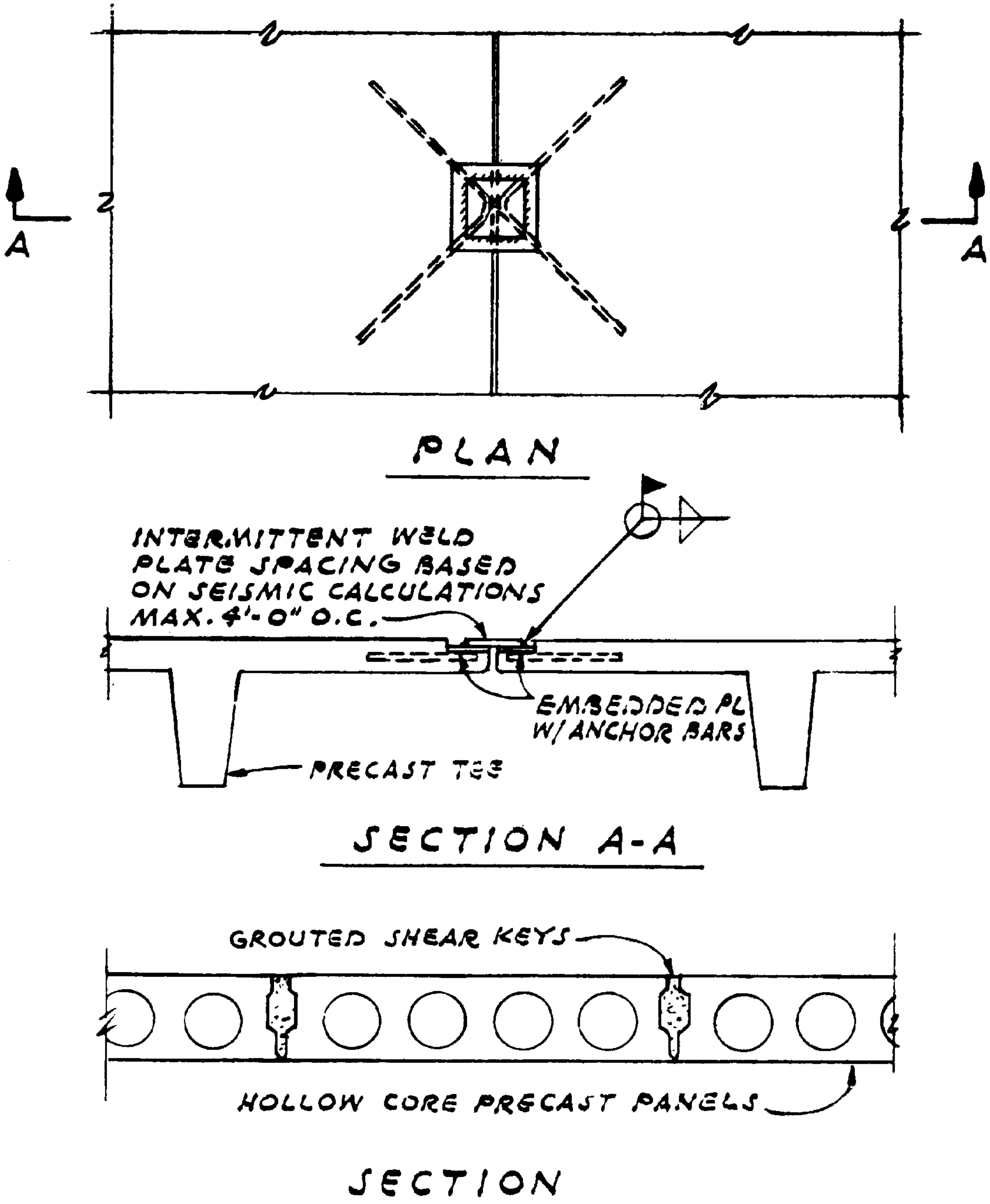



*Figure 5-12. Concrete diaphragms using precast units—details permitted in Seismic Zone 1 only.*

for field attachment of steel decks are shown in figure 5-16. In addition to those shown, standard fillet ( C -inch by 1-inch) and butt welds are also used. The depth of deck units will not be less than 1½ inches.

(2) *Definitions of special symbols.* Definitions of the special symbols used in the determination of the working shears and flexibility of steel deck diaphragms are as follows—

- $a$ = number of seam attachments in span $L_V$ along a seam
- $a_p$ = average spacing of profile channel closures, in feet
- $a_s$ = center-to-center spacing of seam welds in feet, usually $L_v/a$
- $a_w$ = spacing of marginal welds in feet
- $b$ = width of deck unit in feet
- $C_1$ = 1
- $C_2$ = 1 for button-punched seams; $40t_s^{1/2}1'_w$ for welded seams
- $C_3$ = 1 for button-punched seams; $150t_s1'_w$ for welded seams
- $C_4$ = 1 for button-punched seams; $6/L_v$ for welded seams
- $C_5$ = 1.2 for continuous angle closure; 1 for continuous zee closure; $1.44/a_p$ for profile channel closure

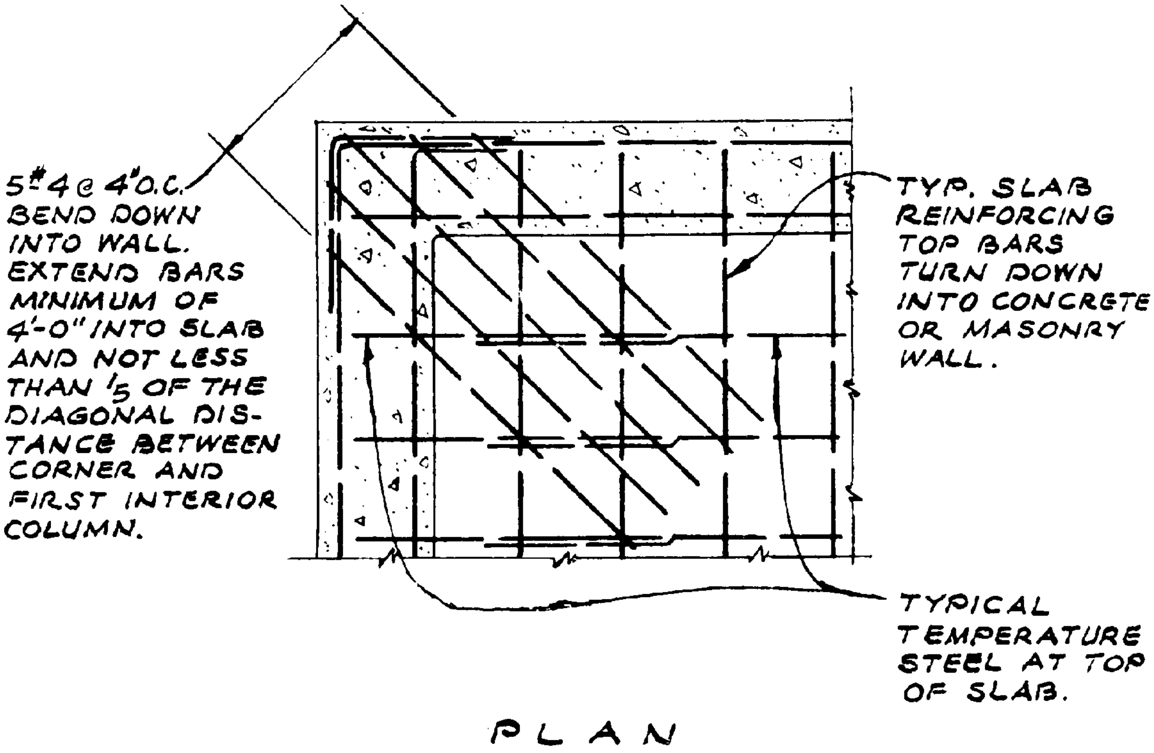



*Figure 5-13. Corner of monolithic concrete diaphragm.*

$d$ = distance in feet between outermost puddle welds attaching a deck unit to the supporting framing member

$F_1, F_2 ...$ = components contributing to the flexibility factor $F = \sum F_n$

$f'c_c$ = compressive strength of fill concrete at 28 days in pounds per square inch

$h$ = height of fluted elements in inches (1½ inch minimum)

$I_D$ = gross moment of inertia of deck unit about vertical centerline axis through unit, in inches to the fourth power

$I_x$ = gross moment of inertia of deck unit about the horizontal neutral axis of the deck cross-section per foot of width, in inches to the fourth power

$L_1$ = distance in feet between vertical resisting element (such as shear wall) and the point to which the deflection is to be determined

$L_2$ = average length of each deck unit in feet

$1_3$ = length of edge lip on deck panel in inches (see detail G in fig 5-16)

$l_3$ = distance in feet between shear transfer elements

$L_R$ = vertical load span of deck units in feet

$L_v$ = minimum length in inches of seam weld

$l'_w$ = effective length in inches of seam weld; the ratio of $1'_w/1_w$ for the various types of seam welds is given in figure 5-16

$n$ = average number of vertical deck elements per foot which are laterally restrained at the bottom by puddle welds

$q_D$ = working shear in pounds per foot; the one-third increase usually permitted on working stresses is not applicable to this value

$q_1, q_2 ..$ = components or limiting values of working shear in pounds per foot

$q_{ave}$ = average shear in diaphragm over length $L_1$ in pounds per foot

$R$ = $L_V/L_2$

$S$ = section modulus in feet of puddle weld group at supports (each weld assumed as unit area)

$t_1$ = thickness of flat sheet elements in inches (22-gauge minimum)

$t_2$ = thickness of fluted element in inches (22-gauge minimum)

$t'_2$ = effective thickness of fluted elements in inches; see figure 5-16 for ratio of $t'_2/t_2$

$t_c$ = thickness of closure element in inches

$f_f$ = thickness of fill over top of deck in inches

$t_s$ = thickness in inches of deck sheet at seams

$w$ = unit weight of fill concrete in pounds per cubic foot

(3) Connections at ends and at supporting beams. Refer to Type A and Type B details, paragraphs 5-9*b* and 5-9*c*.

(4) Connections at marginal supports. Marginal welds for all types of steel deck diaphragms will be spaced as follows—

$$a_w = \frac{35{,}000(t_1 + t_2')C_1}{q} \quad \text{for puddle welds} \qquad \text{(eq 5-6)}$$

$$a_w = \frac{1{,}200_w'}{q} \quad \text{for fillet welds and seam welds} \qquad \text{(eq 5-7)}$$

In no case will the spacing be greater than 3 feet. See figure 5-17.

(5) *Nonwelded fasteners.* Fastening methods other than welds—such as self-drilling, powder-actuated, or pneumatically driven fasteners—may be used provided that equivalence to the welded

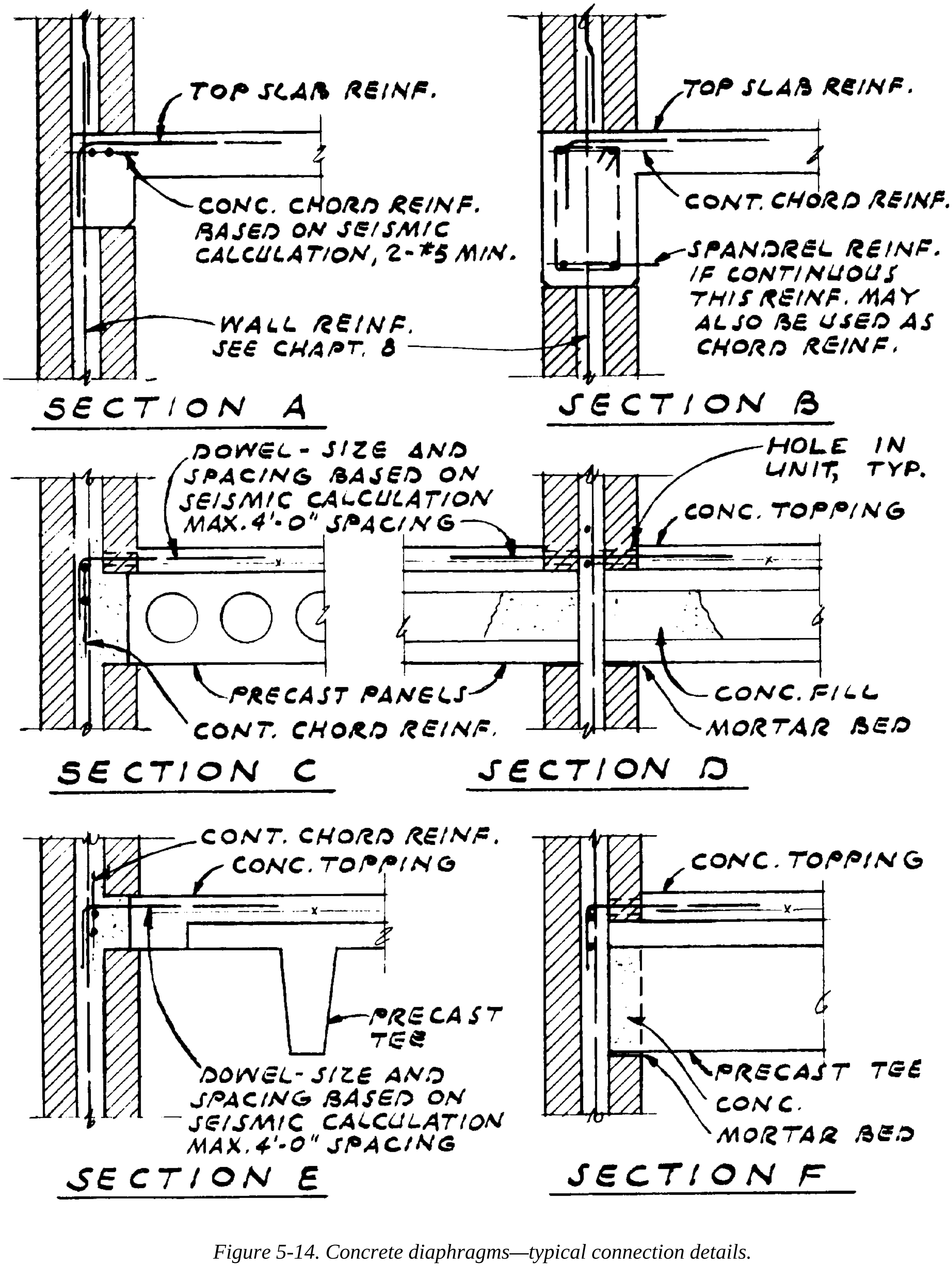



*Figure 5-14. Concrete diaphragms—typical connection details.*

method can be shown by approved test data. The results of such test data will be presented by means of equations or tables for $q_D$ and F in a manner similar to that used in paragraphs 5-9*b*, 5-9*c*, and 5-9*d*.

(6) *Maximum effective thicknesses and weld lengths*. Even though greater thicknesses and weld lengths may be installed, the maximum values for use in determining the working shears in each type of diaphragm will be as follows:

$t_1 = t_2 = t_3 = 0.060$ inch

$t_c = 0.075$ inch

$l_w = 2$ inches

(7) *Thickness of steel.* The thickness of steel

| Class | Compressive Strength | Poured Gypsum Thickness | Welded Wire Fabric | *ALLOWABLE SHEAR VALUES ($q_D$) | |
|---|---|---|---|---|---|
| | | | | Bulb Tees | Trussed Tees |
| A | 500 | 2½" | 4 x 8<br>#12 - #14 | Not Allowed | 890 |
| A | 500 | 2½" | 6 x 6<br>W1.4 x W1.4 | Not Allowed | 1,040 |
| B | 1,000 | 2½" | 4 x 8<br>#12 - #14 | 1,040 | 1,040 |
| B | 1,000 | 2½" | 6 x 6<br>W1.4 x W1.4 | 1,140 | 1,140 |

NOTE: *1/3 increase usually permitted on working stresses in seismic design not applicable.

*Table 5–3. Shear values of poured gypsum diaphragms.*

| Bolt or Dowel Size (Inches) | Embedment (Inches) | Shears (Pounds) |
|---|---|---|
| 3/8 Bolt | 5 | 250 |
| 1/2 Bolt | 5 | 350 |
| 5/8 Bolt | 5 | 500 |
| 3/8 Deformed Dowel | 6 | 250 |
| 1/2 Deformed Dowel | 6 | 350 |

Notes: *1/3 increase usually permitted on working stresses in seismic design is not applicable.

See Details A2 and A3 in Figure 5-15.

*Table 5–4. Shear on anchor bolts and dowels—reinforced gypsum concrete.*

| | |
|---|---|
| Class A | 840 pounds per foot |
| Class B | 1,140 pounds per foot |

Notes: *1/3 increase usually permitted on working stresses in seismic design is not applicable.

See Details A2, A4, and A5 in Figure 5-15.

*Table 5–5. Maximum shear on trussed tees.*

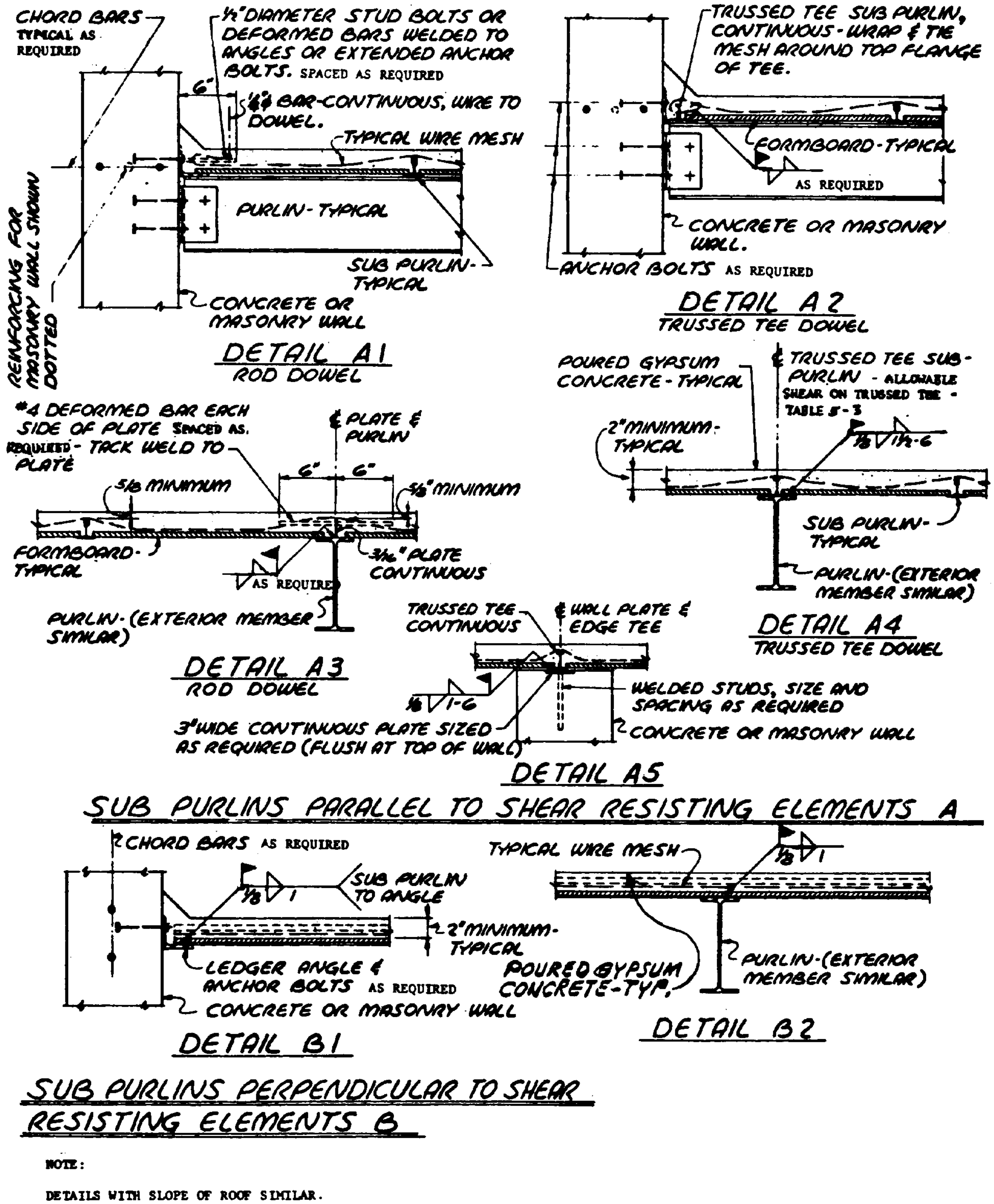
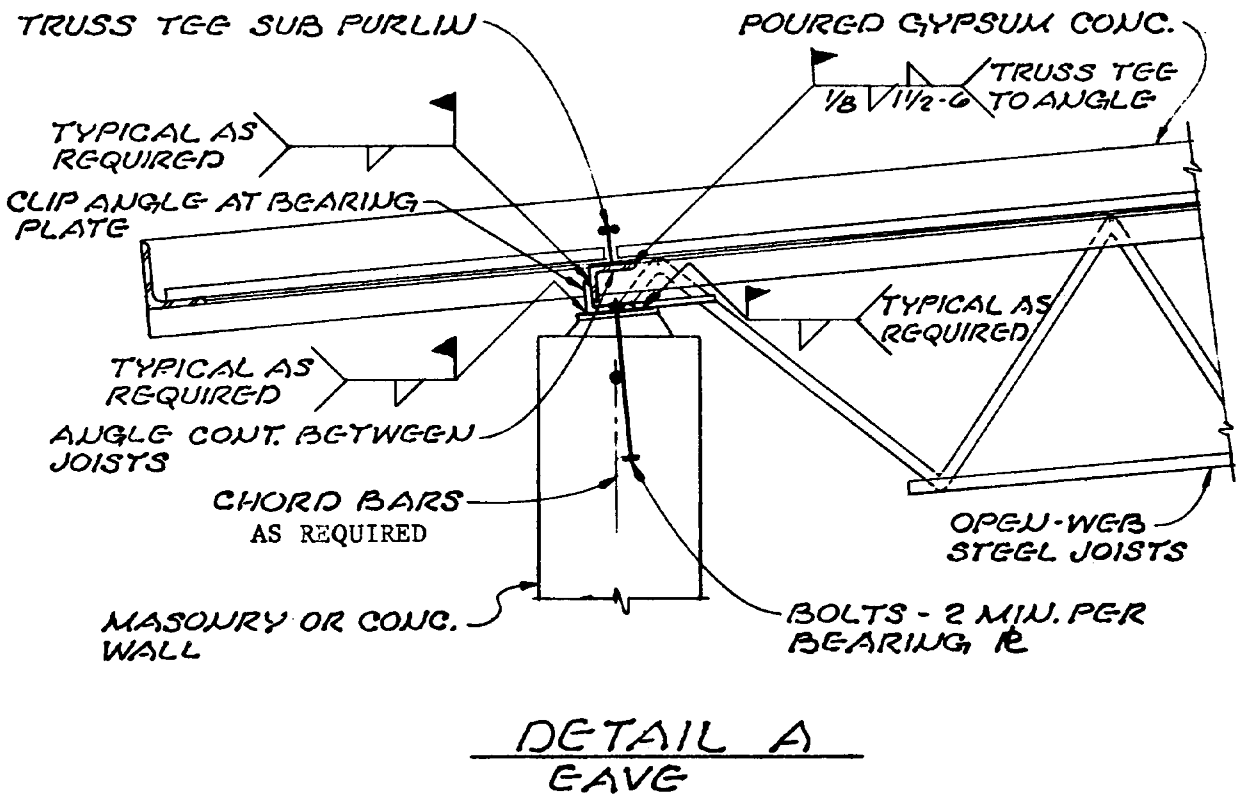



*Figure 5-15. Poured gypsum diaphragms—typical details.*



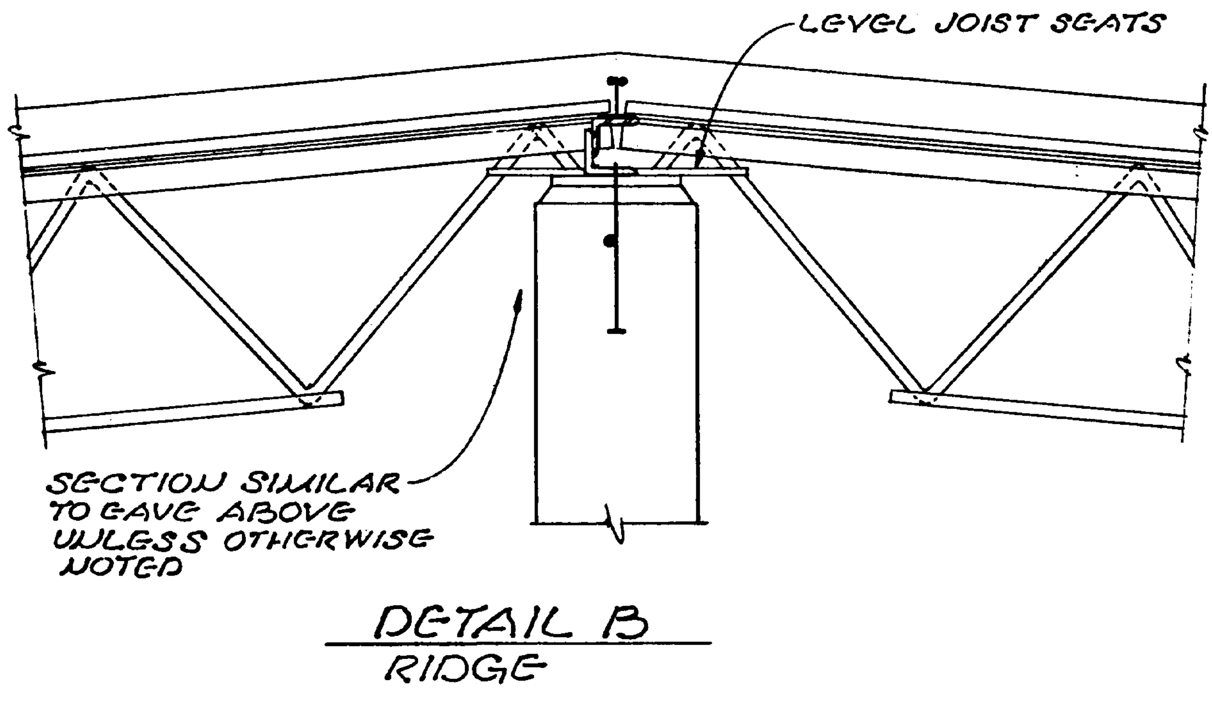




*Figure 5-15. Continued.*

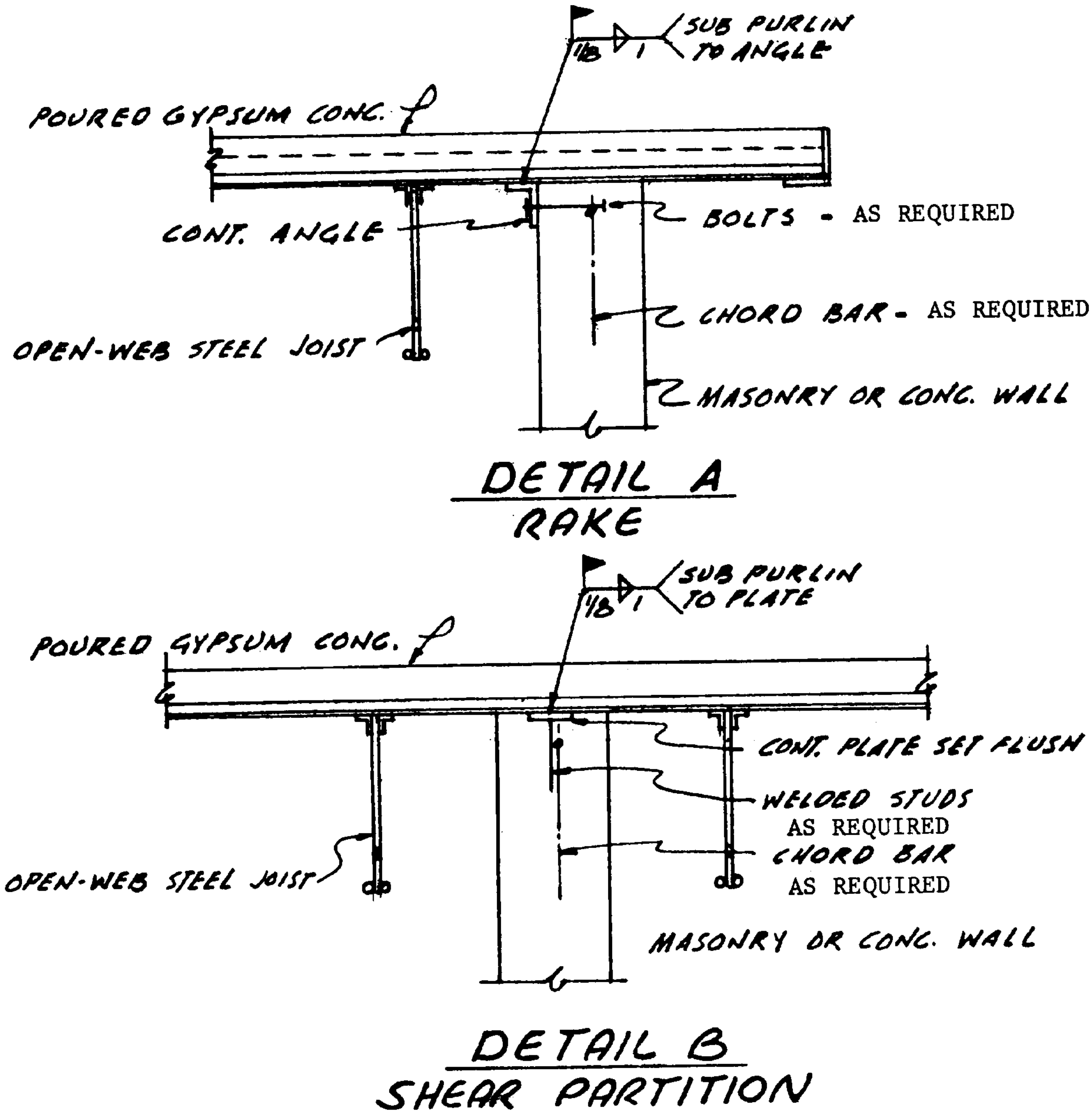



*Figure 5-15. Continued.*

before coating with paint or galvanizing shall be in accordance with the following table. The thickness of the uncoated steel shall not at any location be less than 95% of the design thickness.

| *Gauge* | *Design Thickness* | *Minimum Thickness* |
|---|---|---|
| 22 | 0.0295 | 0.028 |
| 20 | 0.0358 | 0.034 |
| 18 | 0.0474 | 0.045 |
| 16 | 0.0598 | 0.057 |

*b. Type A diaphragms—decks having shear transfer elements directly attached to framing.* Mutliple-plate steel decks with the flat element adjacent to framing members and single-plate steel decks fall into this category of diaphragms when each deck unit is attached to the framing by at least two puddle welds or equivalent fasteners, as described in figure 5-16. Thicknesses $t_1$, $t_2$, and $t_s$ will not be less than 22 gauge. Seam attachments will be made at least at midspan of $L_v$, but the spacing of attachments between supports will not exceed 3 feet on center. Typical details of Type A diaphragms and attachments are shown in figure 5-18.

(2) *Shear capacity.* The working shear will be limited to that determined by the following formulas—

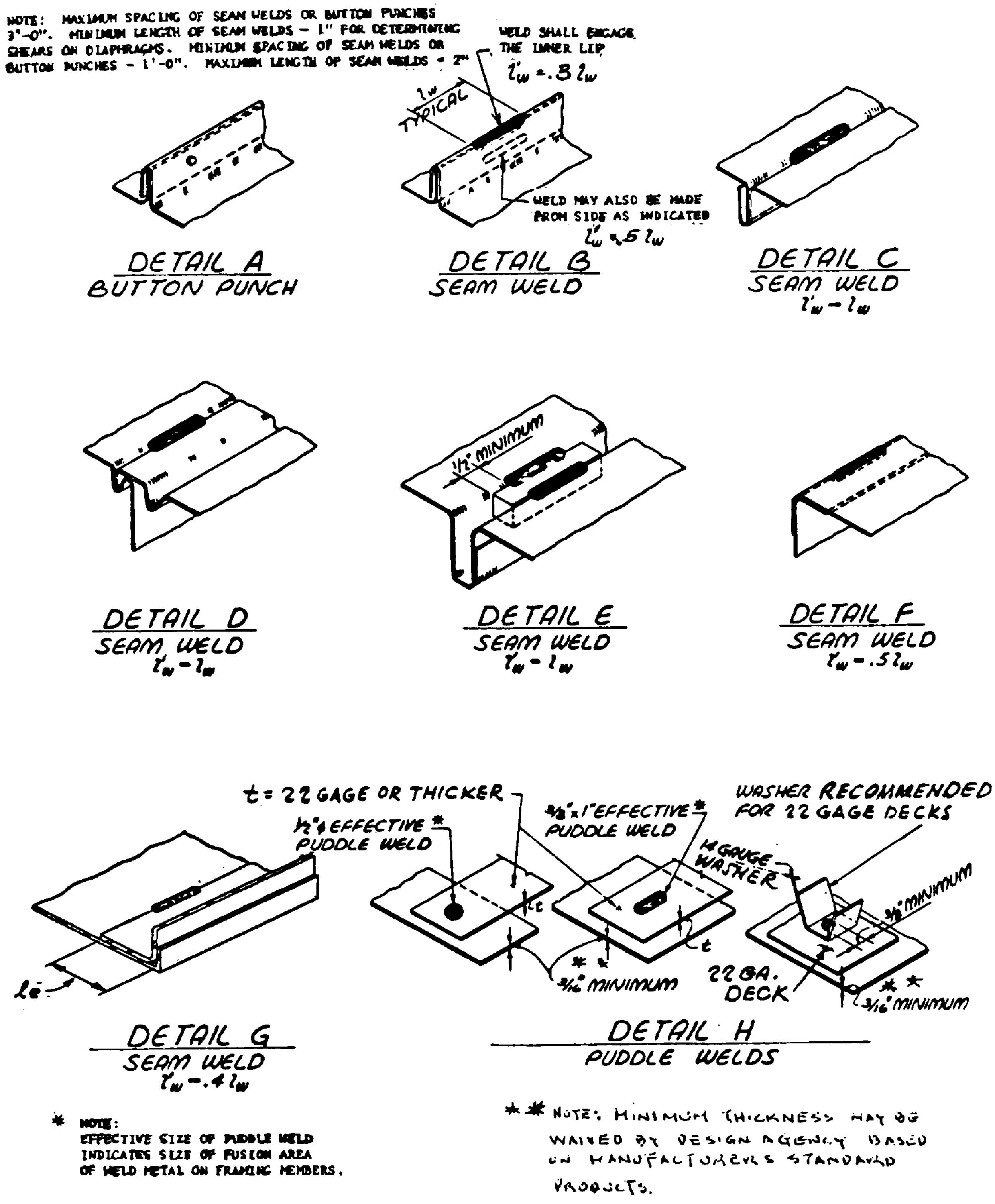



*Figure 5-16. Steel deck diaphragms—typical details of fastenings.*

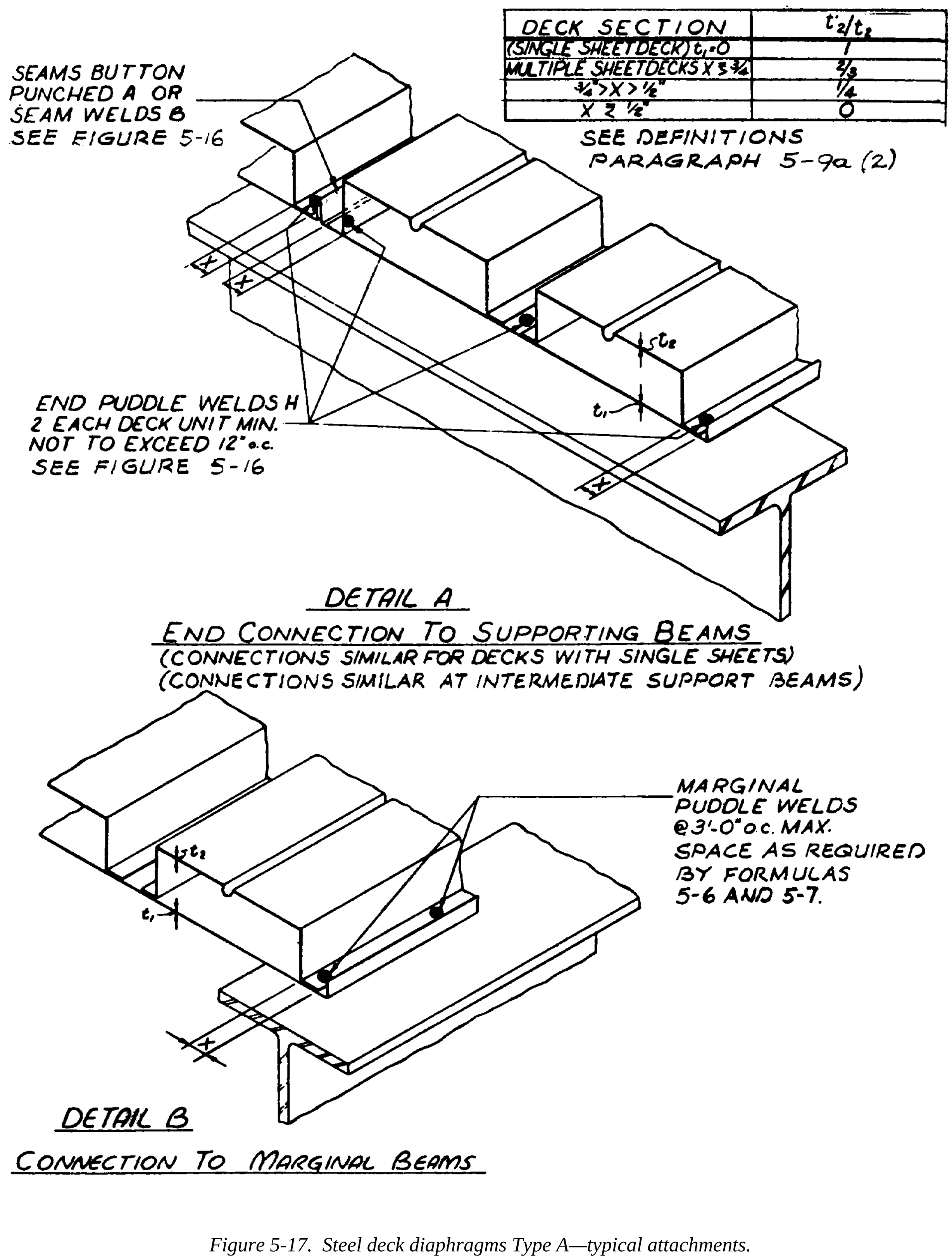



*Figure 5-17. Steel deck diaphragms Type A—typical attachments.*

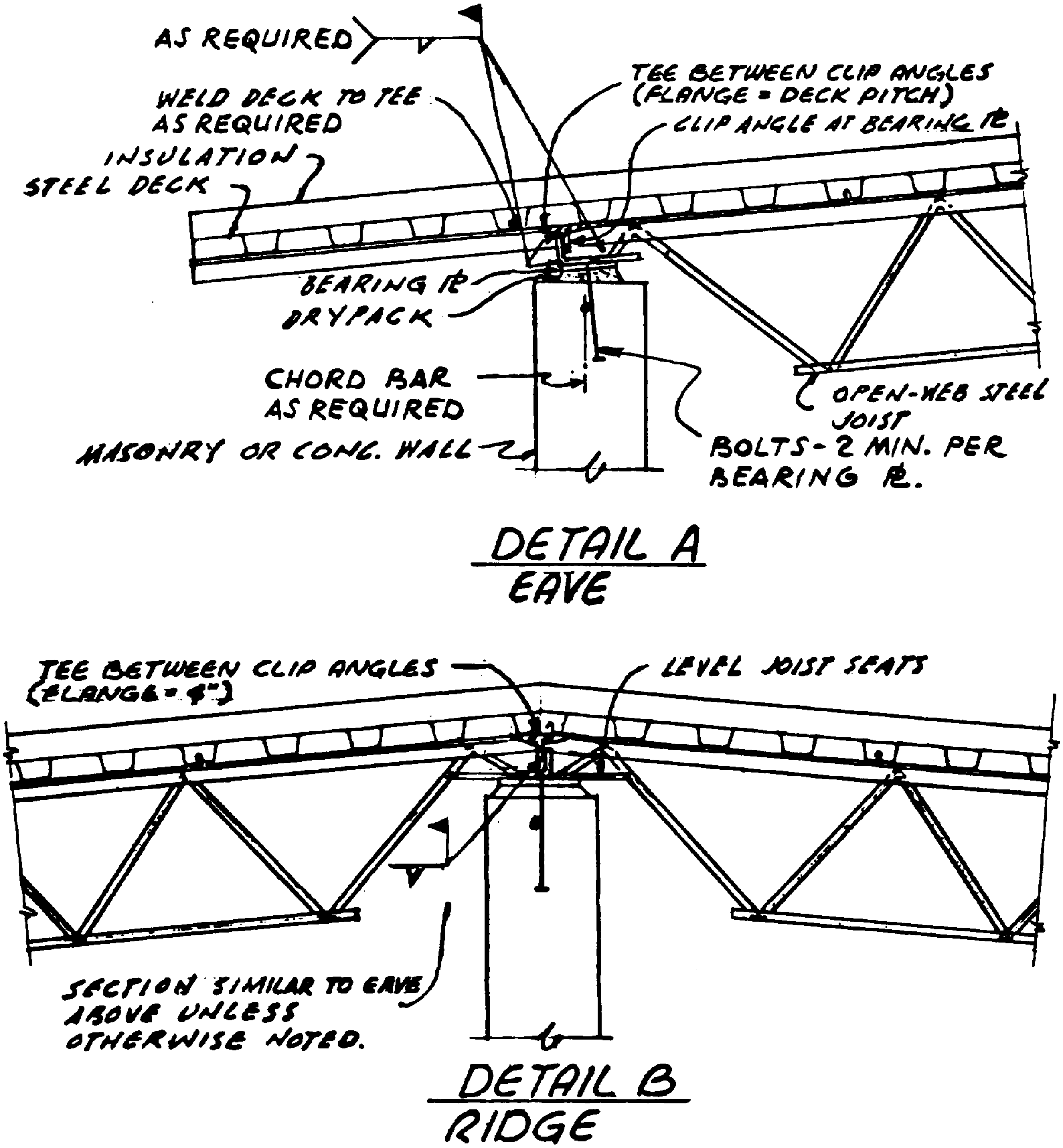



*Figure 5-18. Steel deck diaphragm. Type A—typical details with open-web joists.*

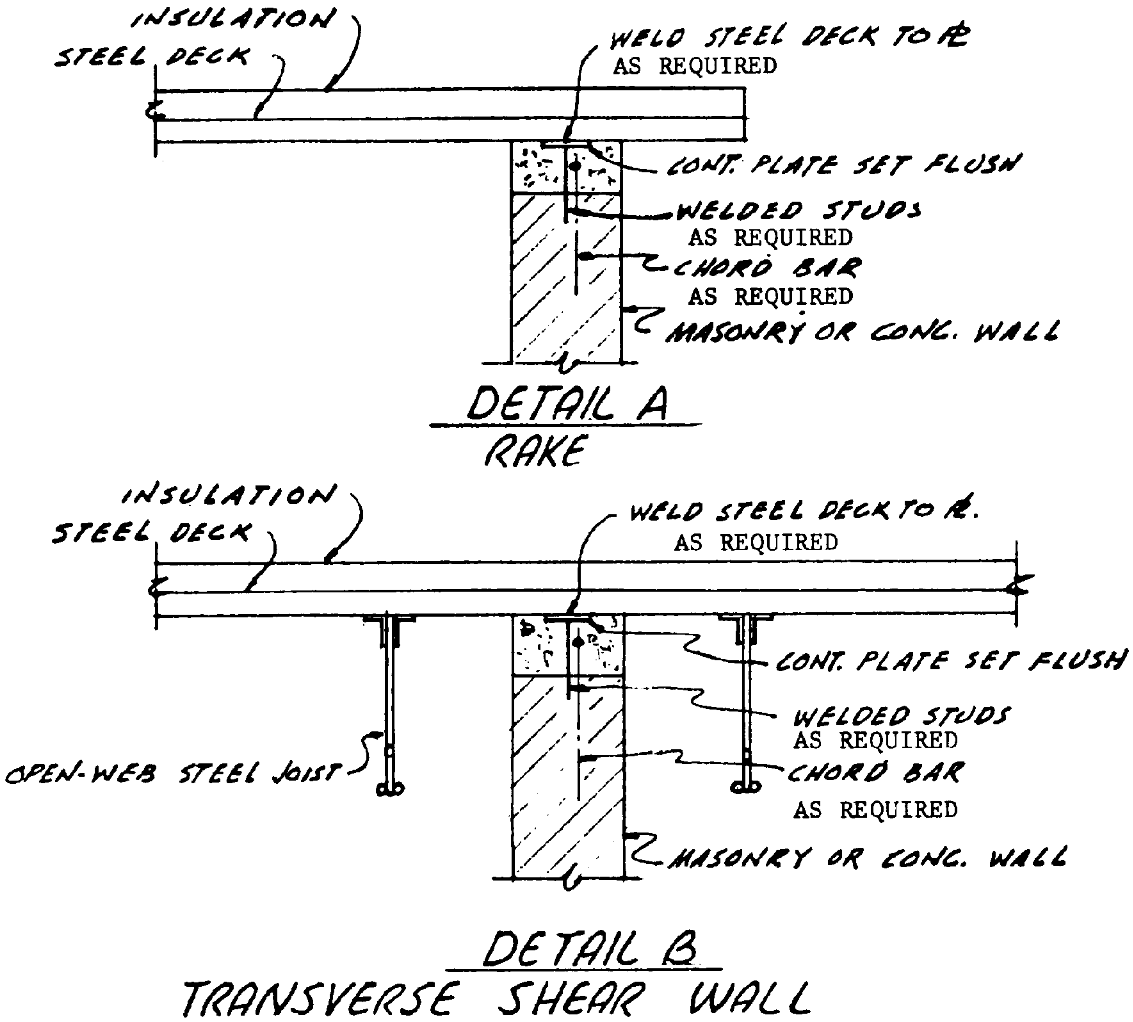



*Figure 5-18. Continued*

$$q_D = (q_1 + q_2)\,\frac{q_3}{q_2} \qquad \text{(eq 5-8)}$$

where $q_3/q_2 \leq C_1$, but $q_D$ is not to exceed

$$\frac{I_x \times 10^6}{2L_v^2} \qquad \text{(eq 5-9)}$$

nor

$$\frac{10^4}{1.5\sqrt{L_v\left(F_1 + F_2 + \frac{F_3 L_2}{12}\right)}} \qquad \text{(eq 5-10)}$$

Equation 5-10 applies only when $1_e < ½$ inch; refer to Detail G in figure 5-16.

$$q_1 = \frac{92S(t_1 + t_2')K}{bL_v} \qquad \text{(eq 5-11)}$$

where

$$K = \frac{1{,}000}{\left[1 + S\frac{1}{\left[\frac{(t_1 + t_2)t_1}{t_2^2} + 100n^{1/2}t_2^2\sqrt{\frac{43}{h}}\left(\frac{t_2}{t_1 + t_2}\right)^3\right]^2}\right]^{1/2}} \qquad \text{(eq 5-12)}$$

$$q_2 = \frac{abt_2^{1/2}C_2}{2}\left[q_1\left[\frac{500}{I_D} + \frac{1}{L_v dS(t_1 + t_2')^2}\right]\right]^{1/2} \qquad \text{(eq 5-13)}$$

$$q_3 = \frac{3600t_s aC_3}{L_v} \qquad \text{(eq 5-14)}$$

(2) Flexibility factor. The flexibility factor, F, will be determined by the following formulas:

$$F = F_1 + F_2 + F_3 \qquad \text{(eq 5-15)}$$

where

$$F_1 = \frac{1}{12(t_1 + t_2)} \qquad \text{(eq 5-16)}$$

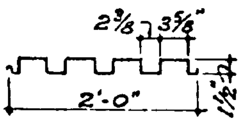
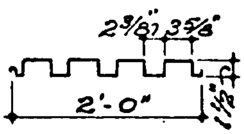
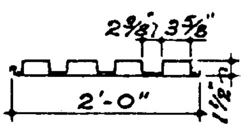
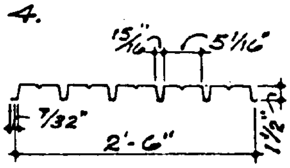
$$F_2 = \frac{bL_v^2C_4}{160}\left[\frac{500}{I_D} + \frac{1}{L_v dS(t_1 + t_2)}\right]\frac{q_1}{q_1 + q_2} \qquad \text{(eq 5-17)}$$

$$F_3 = \frac{R}{L_v\left(t_1 + \frac{12.5n^2C_1^2t_n^3}{h}\right)} \qquad \text{(eq 5-18)}$$

The flexibility of these diaphragms will vary within a wide range. Arrangements can be used that fall into the semirigid, semiflexible, and flexible categories.

(3) Sample calculations and tables. Summaries of allowable shear ($q_D$) and flexibility factors (F) for some of the more common cross sections are shown in figure 5-19. Sample calculations using the formulas for these cross sections are given in figure 5-20.

TABLE OF ALLOWABLE SHEAR ($q_D$) AND FLEXIBILITY FACTOR (F)

| SECTION | Welds * | SEAM FASTENING | GAGE | | SPAN ($L_V$) 4'-0" | 5'-0" | 6'-0" | 7'-0" | 8'-0" | 9'-0" | 10'-0" |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1.  | 3 | ** BUTTON PUNCH @ 24" o.c. | 16 | $q_D$ | 1260 | 1030 | 870 | 760 | 680 | 620 | 560 |
| | | | | F | 5.7+ 24.7R | 7.0+ 27.8R | 8.3+ 23.1R | 9.6+ 19.8R | 11+ 17.4R | 12+ 15.4R | 14+ 13.9R |
| | | | 18 | $q_D$ | 900 | 740 | 630 | 550 | 500 | 450 | 410 |
| | | | | F | 8.1+ 67.8R | 9.9+ 54.2R | 12+ 45.2R | 13+ 38.7R | 15+ 33.9R | 17+ 30.1R | 19+ 27.1R |
| | | | 20 | $q_D$ | 520 | 430 | 370 | 320 | 290 | 260 | 240 |
| | | | | F | 13+ 161R | 15+ 129R | 18+ 107R | 21+ 91.9R | 23+ 80.4R | 26+ 71.4R | 28+ 64.3R |
| | | | 22 | $q_D$ | 340 | 280 | 240 | 210 | 190 | 180 | 160 |
| | | | | F | 17+ 278R | 20+ 222R | 23+ 185R | 27+ 159R | 30+ 139R | 32+ 123R | 35+ 111R |
| 2.  | 5 | ** BUTTON PUNCH @ 24" o.c. | 16 | $q_D$ | 1650 | 1340 | 1130 | 980 | 870 | 790 | 720 |
| | | | | F | 5.0+ 8.68R | 6.1+ 6.94R | 7.3+ 5.79 R | 8.5+ 4.96R | 9.8+ 4.34R | 11+ 3.86R | 13+ 3.47R |
| | | | 18 | $q_D$ | 1220 | 990 | 840 | 730 | 660 | 580 | 520 |
| | | | | F | 7.1+ 17.0R | 8.6+ 13.6R | 10+ 11.3R | 12+ 9.69R | 14+ 8.48R | 15+ 7.54R | 17+ 6.78R |
| | | | 20 | $q_D$ | 700 | 560 | 470 | 410 | 360 | 320 | 290 |
| | | | | F | 11+ 40.2R | 13+ 32.1R | 16+ 26.8R | 18+ 23.0R | 21+ 20.1R | 23+ 17.9R | 26+ 16.1R |
| | | | 22 | $q_D$ | 450 | 370 | 310 | 270 | 240 | 220 | 200 |
| | | | | F | 15+ 69.4R | 18+ 55.5R | 21+ 46.3R | 24+ 39.7R | 27+ 34.7R | 30+ 30.9R | 32+ 27.8R |
| 3.  | 3 | ** BUTTON PUNCH @ 24" o.c. | 18-18 | $q_D$ | 1563 | 1265 | 1068 | 927 | 822 | 741 | 676 |
| | | | | F | 3.1 + 4.84 R | 3.8 + 3.87 R | 4.6 + 3.22 R | 5.5 + 2.76 R | 6.5 + 2.42 R | 7.5 + 2.15 R | 8.6 + 1.93 R |
| | | | 16-16 | $q_D$ | 1968 | 1590 | 1340 | 1161 | 1028 | 925 | 842 |
| | | | | F | 2.2 + 3.72 R | 2.8 + 2.98 R | 3.4 + 2.48 R | 4.1 + 2.13 R | 4.8 + 1.86 R | 5.6 + 1.65 R | 6.5 + 1.49 R |
| | | | 16-18 | $q_D$ | 1911 | 1545 | 1302 | 1129 | 1000 | 900 | 820 |
| | | | | F | 2.5 + 3.95R | 3.1 + 3.14 R | 3.8 + 2.62R | 4.5 + 2.24 R | 5.3 + 1.96R | 6.1 + 1.74 R | 7.0 + 1.57 R |
| | | | 20-20 | $q_D$ | 1168 | 948 | 792 | 673 | 585 | 517 | 463 |
| | | | | F | 4.7 + 6.68R | 5.8 + 5.34 R | 7.0 + 4.45R | 8.3 + 3.82 R | 9.7 + 3.34R | 11.1 + 2.97 R | 12.6 + 2.67 R |
| 4.  | 6 | 1½" SEAM WELD @ 18" o.c. | 18 | $q_D$ | 990 | 890 | 820 | 760 | 710 | 680 | 650 |
| | | | | F | 5.7+ 17.0R | 5.5+ 13.6R | 5.4+ 11.3R | 5.3+ 9.68R | 5.2+ 8.47R | 5.1+ 7.53R | 5.0+ 6.78R |
| | | | 20 | $q_D$ | 710 | 640 | 590 | 550 | 520 | 490 | 460 |
| | | | | F | 8.5+ 40.2R | 8.1+ 32.1R | 7.8+ 26.8R | 7.6+ 23.0R | 7.3+ 20.1R | 7.1+ 17.9R | 6.9+ 16.1R |
| | | | 22 | $q_D$ | 480 | 420 | 380 | 350 | 330 | 310 | 300 |
| | | | | F | 11+ 69.4R | 10+ 55.5R | 9.7+ 46.3R | 9.3+ 39.7R | 8.9+ 34.7R | 8.6+ 30.9R | 8.3+ 27.8R |

5. See Figure 5-19, sheet 2 of 2

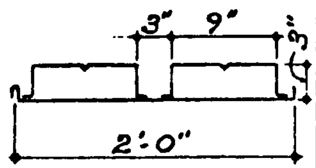
** SEAM WELDS ARE PREFERABLE.

*Number of welds at end and at intermediate support beams.

NOTE:
THE GAGES FOR MULTIPLE SHEET DECKS ARE DESIGNATED WITH THE GAGE OF THE FLAT SHEET FIRST AND FLUTED SHEET SECOND.

*Figure 5-19. Steel deck diaphragm Type A—allowable shears and flexibility factors.*

TABLE OF ALLOWABLE SHEAR ($q_D$) AND FLEXIBILITY FACTOR (F)

| SECTION | END WELDS | SEAM FASTENING | GAGE | | SPAN (Lv) 4'-0" | 5'-0" | 6'-0" | 7'-0" | 8'-0" | 9'-0" | 10'-0" | 11'-0" | 12'-0" | 13'-0" |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5.  | 3 | BUTTON PUNCH @ 24" o.c. | 18-18 | $q_D$ | 1180 | 960 | 810 | 710 | 630 | 570 | 520 | 480 | 450 | 420 |
| | | | | F | 3.0+ 5.01R | 3.7+ 4.01R | 4.5+ 3.34R | 5.4+ 2.87R | 6.3+ 2.51R | 7.3+ 2.23R | 8.3+ 2.01R | 9.3+ 1.82R | 10.4+ 1.67R | 11.6+ 1.54R |
| | | | 16-16 | $q_D$ | 1480 | 1200 | 1010 | 880 | 780 | 700 | 640 | 590 | 550 | 520 |
| | | | | F | 2.1+ 3.93R | 2.6+ 3.14R | 3.2+ 2.62R | 3.8+ 2.25R | 4.5+ 1.97R | 5.2+ 1.75R | 5.9+ 1.57R | 6.7+ 1.43R | 7.5+ 1.31R | 8.4+ 1.21R |
| | | | 18-16 | $q_D$ | 1160 | 940 | 800 | 690 | 620 | 560 | 510 | 470 | 440 | 410 |
| | | | | F | 2.5+ 4.84R | 3.1+ 3.87R | 3.8+ 3.22R | 4.5+ 2.77R | 5.3+ 2.42R | 6.2+ 2.15R | 7.0+ 1.94R | 7.9+ 1.76R | 8.9+ 1.61R | 9.9+ 1.49R |
| | | | 16-18 | $q_D$ | 1510 | 1230 | 1040 | 900 | 800 | 720 | 660 | 610 | 570 | 530 |
| | | | | F | 2.5+ 4.04R | 3.1+ 3.23R | 3.7+ 2.69R | 4.4+ 2.31R | 5.1+ 2.02R | 5.9+ 1.80R | 6.7+ 1.62R | 7.6+ 1.47R | 8.5+ 1.35R | 9.4+ 1.24R |

NOTE:
THE GAGES FOR MULTIPLE SHEET DECKS ARE DESIGNATED WITH THE GAGE OF THE FLAT SHEET FIRST AND FLUTED SHEET SECOND.

*Figure 5–19. Continued.*

SAMPLE CALCS. NO.1 FOR TYPE A DIAPHRAGM.

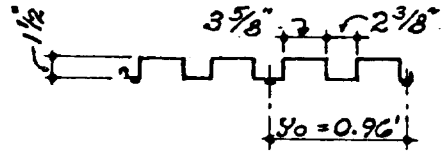
$$q_D = (q_1 + q_2)\frac{q_3}{q_2} \qquad \text{(PARA. 5-9b)}$$

$$q_1 = \frac{92\,S(t_1 + t'_2)K}{b L_v}$$

$$q_2 = \frac{a b t_s^{1/2} C_2}{2}\left\{q_1\left[\frac{500}{I_D} + \frac{1}{L_v d S(t_1 + t'_2)^2}\right]\right\}^{1/2}$$

$$K = \frac{1000}{\left\{1 + S\left[\dfrac{1}{\dfrac{(t_1+t_2)t_1}{t_2^2} + 100 n^{1/2} t_2^2 \sqrt{\dfrac{43}{n}}\left(\dfrac{t_2}{t_1+t_2}\right)^3}\right]^2\right\}^{1/2}}$$

$$q_3 = \frac{3600\, t_s\, a\, C_3}{L_v}$$




20 GAGE SINGLE PLATE DECK, BUTTON PUNCH SEAMS @ 24" o.c., 3 END WELDS

$t_1 = 0$

$t_2 = t'_2 = t_s = 0.036"$

$S = \dfrac{\Sigma y^2}{y_0} = \dfrac{2 \times .96^2}{.96} = 1.92'$

$I_x = .23$

$b = 2'$ $\quad h = 1.5"$

$L_v = 10'\text{-}0"$ $\quad a = L_v/2$

$n = 4/2 = 2$ $\quad d = 1.92 = 2y_0$

$I_D = 68$

$C_1 = C_2 = C_3 = C_4 = 1$

$$K = \frac{1000}{\left\{1 + 1.92\left[\dfrac{1}{100\sqrt{2}\,(.036)^2\sqrt{\dfrac{43}{1.5}}}\right]^2\right\}^{1/2}} = \frac{1000}{1.73} = 578$$

$$q_1 = \frac{92 \times 1.92 \times .036 \times 578}{2 \times 10} = 184$$

$$q_2 = \frac{5 \times 2\sqrt{.036}}{2}\left\{184\left[\frac{500}{68} + \frac{1}{10 \times 1.92 \times 1.92 \times (.036)^2}\right]\right\}^{1/2}$$

$$= .945\sqrt{5250} = 68.4$$

$$q_3 = \frac{3600 \times .036 \times 5}{10} = 64.8 \qquad \frac{q_3}{q_2} = \frac{64.8}{68.4} = 0.95$$

$$q_D = (184 + 68.4)\,.95 = 240$$

$$\frac{I_x \times 10^6}{2 L_v^2} = \frac{.23 \times 10^6}{2 \times 10^2} = 1150 > 240 \ \underline{\text{O.K.}}$$

$q_D = 240$ (FIGURE 5-19: $L_v = 10'$, 20 ga.)

$$F = F_1 + F_2 + F_3$$

$$F_1 = \frac{1}{12(t_1 + t_2)} = \frac{1}{12 \times .036} = 2.32$$

$$F_2 = \frac{b L_v^2 C_4}{160}\left[\frac{500}{I_D} + \frac{1}{L_v d S (t_1 + t'_2)^2}\right]\frac{q_1}{q_1 + q_2} = \frac{2 \times 100}{160}\left[7.35 + 21.1\right]\frac{184}{252.4} = 25.9$$

$$F_3 = \frac{R}{L_v\left(t_1 + \dfrac{12.5 n^2 t_2^3 C_1^2}{h}\right)} = \frac{R}{10\left(\dfrac{12.5 \times 4 \times .036^3}{1.5}\right)}$$

$$= \frac{R}{10 \times .00156} = 64R$$

$$F = 2.32 + 25.9 + 64R = 28.2 + 64R$$

(SEE FIGURE 5-19: $L_v = 10'$, 20 ga.)

*Figure 5-20. Steel deck diaphragm Type A—sample calculation.*

SAMPLE CALC. NO. 2 FOR TYPE A DIAPHRAGM

18 GAGE SINGLE PLATE DECK.
BUTTON PUNCH SEAMS @ 24" o.c.
5 END WELDS
$L_v$ - 9'-0"

$$K = \frac{1000}{\left\{1+2.44\left[\frac{1}{100\times 2(.048)^2\sqrt{\frac{43}{1.5}}}\right]^2\right\}^{1/2}} = \frac{1000}{1.18} = 845$$

$$q_1 = \frac{92\times 2.44\times .048\times 845}{2\times 9} = 505$$

$$q_2 = \frac{9\times .048^{1/2}}{2}\left\{505\left[\frac{500}{91} + \frac{1}{9\times 1.92\times 2.44(.048)^2}\right]\right\}^{1/2}$$
$$= 87.9$$

$$q_3 = \frac{3600\times .048\times 4.5}{9} = 86.5 \quad \frac{q_3}{q_2} = \frac{86.5}{87.9} = .985$$

$$q_D = (505+87.9).985 = \underline{584}$$

$$\frac{I_x\times 10^6}{2L_v^2} = \frac{.34\times 10^6}{2\times 9^2} = 2099 > 584 \ \underline{\text{O.K.}}$$

$q_D$ = 580 (FIGURE 5-19: $L_V$ = 9', 18 ga.)

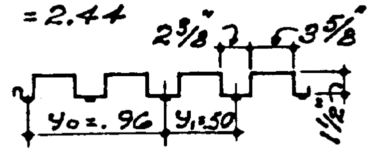
$$F_1 = \frac{1}{12\times .048} = 1.74$$

$$F_2 = \frac{2\times 9^2}{160}(15.8)\frac{505}{592.9} = 13.7$$

$$F_3 = \frac{R}{9\left(\frac{12.5\times 16(.048)^3}{1.5}\right)} = \frac{R}{9\times .0147} = 7.55R$$

$F = 1.74 + 13.7 + 7.55R = \underline{15.4 + 7.6R}$
(SEE FIGURE 5-19: $L_V$ = 9', 18 ga.)

$t_1 = 0$
$t_2 = t_2' = t_s = .048"$
$$S = \frac{\Sigma y^2}{y^o} = \frac{2\times .96^2 + 2\times .50^2}{.96} = 2.44$$




$b = 2'$ $\quad h = 1.5"$
$L_v = 9'$ $\quad I_x = .34$
$n = 8/2 = 4$ $\quad a = L_v/2$
$I_D = 91$ $\quad d = 1.92 = 240$
$C_1 = C_2 = C_3 = C_4 = 1$

*Figure 5-20. Continued.*

*c. Type B diaphragms.* These are decks having an elevated plate of shear transfer. Multiple steel decks with fluted elements adjacent to framing members and single-plate steel decks with fluted elements incapable of being welded to framing with at least two puddle welds or equivalent fasteners per unit fall into this category of diaphragm. This type of diaphragm has only welded seam attachments. The units will be composed of sheets not less than 20 gauge. Seam attachment spacing will not exceed 3 feet on center. Typical details of Type B diaphragms and attachments are shown in Figure 5-2 1.

(1) *Shear capacity.* The working shear will be limited to that determined by the following formulas—

$q_D = q_3$, $q_4$, or $q_5$ whichever is lesser,
but not to exceed 1,050 pounds per foot. (eq 5-19)

$$q_3 = \frac{0.6t_2^2al_w'}{L_v} \qquad \text{(eq 5-20)}$$

$$q_4 = \frac{t_s}{10}\left(\frac{1}{a_s}\right)^2\times 10^6 \qquad \text{(eq 5-21)}$$

$$q5 = \frac{C_5t_c^2\times 10^6}{2h^{1/2}} \qquad \text{(eq 5-22)}$$

(2) *Flexibility factor.* The flexibility factor, F, will be determined by the following formulas—

$$F_1 = F_1 + F_4 + F_5 \qquad \text{(eq 5-23)}$$

where

$$F_1 = \frac{1}{12(t_1+t_2)} \qquad \text{(eq 5-24)}$$

$$F_5 = \frac{20{,}000}{L_Rq_5} \qquad \text{(eq 5-25)}$$

$$F5 = \frac{20{,}000}{L_Rq_5} \qquad \text{(eq 5-26)}$$

These diaphragms will fall into the semirigid and semiflexible categories.